Behavior of slab-column connections under seismic actions

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October 2015

Abstract: This work comprises the study and evaluation of the behavior of slab-column connections. Recent experimental campaigns indicate that the level of gravity loads are a determining factor on the slab’s capacity to accommodate imposed deformation and therefore the maximum drift capacity. One possible way to increase the slab-column connection’s ductility is to use punching shear reinforcement. Results of experimental programs illustrates a very significant enhancement in capacity and deformation resistance of the slab, when compared to the slabs without transverse reinforcement.

Three structures with different lateral stiffness are analyzed to do a design evaluation of parameters such as the drift capacity of the structure and the lateral stiffness itself. Two different approaches of assessing the seismic displacements and the induced forces on the slab-column connection are proposed. The results are discussed and compared with those obtained from an approach where the slab-column connections are considered as secondary seismic member and reinforcement yielding is prevented.

Keywords: Punching shear, slab-column connection, drift capacity, gravity load, seismic action, shear reinforcement, ductility

1. Introduction

Reinforced concrete flat-slabs supported on columns is one of the most commonly used structural system in many countries. The absence of beams in this structural system leads to a particular failure mode, punching shear. One of the dominant parameter for the seismic resistance of flat-slab structures is the rotation capacity of its connections. The absence of beams significantly decreases restrictions on columns rotation at the floors level, leading to a significant increase in deformation, highlighting the importance of the study of the behavior of slab-columns connections under cyclic horizontal actions such as seismic actions. For gravity induced shear forces, punching shear is widely studied. However, slab-column connections subjected to lateral cyclic loads experience very complex behavior that is not well studied. Eurocode 8-1 (EC 8-1) [1] states that slab-column connection must have the capacity to follow the seismically induced displacement of the structure while maintaining its capacity to transfer vertical loads and do not contemplate this system as a primary lateral force resisting system in region of moderate to high seismic risk.
2. Critical shear crack theory and MC 2010

The new fib Model Code for Concrete Structures 2010 (MC 2010) [2] is based on physical models. These models provide physical approaches that explain the role of various parameters and can be considered an evolution of the previous empirical design approaches such as Eurocode 2 (EC 2) [3]. The critical shear crack theory (CSCT) was chosen as the base model to the punching shear provisions [4]. A code strategy based on levels of approximation (LoA) is adopted in MC 2010. This approach is also applicable to punching shear provisions. The LoA approach proposes using theories based on physical models where the number and accuracy of physical parameters can be refined or better estimated by devoting more time to their analysis (see Fig.1)

\[ V_{Rd,c} = k_\psi \sqrt{f_{ck}} b_0 d_v \]  

(2)

Where, \( b_0 \) is the shear resisting control perimeter and \( k_\psi \) is a factor accounting for opening and roughness of shear cracks, whose value can be calculated as:

\[ k_\psi = \frac{1}{1.5 + 0.9 \cdot \psi \cdot d \cdot k_{dg}} \leq 0.6 \]  

(3)

where factor \( k_{dg} \) depends on the maximum aggregate size \( d_g \) and defined as \( k_{dg} = 32/(16 + d_g) \geq 0.75 \).

The CSCT can also be applied to slabs with transverse reinforcement. Various numbers of potential failure modes can develop. Failures modes by punching within the shear reinforced area and crushing of concrete struts are of particular importance. In the first one, the strength depends on both concrete and transverse reinforcement contributions. Transverse reinforcement is activated as the critical shear cracks start opening, increasing the stresses in the reinforcement. The opening of the shear critical cracks leads to a contribution decrease of the concrete [5].

To calculate the punching shear strength, it is necessary to estimate the load-rotation curve, with a general form given by the following equation:

\[ \psi = 1.5 \frac{r_s}{d} \frac{f_{yd}}{E_s} \left( \frac{m_{sd}}{m_{rd}} \right)^{1.5} \]  

(4)

where, \( r_s \) is the distance between column of slab and line of contraflexure of moments, \( f_{yd} \) is the yield strength of flexural reinforcement, \( E_s \) is the modulus of elasticity of flexural steel, \( m_{sd} \) and \( m_{rd} \) are the average acting moment and the average flexural strength, respectively. Four levels of approximation can be used to estimate the load-rotation curve, increasing the degree of accuracy from level I to level IV as shown on Figure 1.

Figure 2 presents the load-rotation relation for punching shear test by Kinnunen and Nylander done on slabs for different reinforcement ratios [6].
The slab behavior for the different reinforcement ratios varied significantly. The horizontal line for $\rho = 0.5\%$ shows the ductile behavior of the slab, with yielding of the entire flexural reinforcement. For intermediate reinforcement ratios, punching shear failure occurs before yielding of the entire flexural reinforcement and for $\rho = 2.0\%$ punching shear failure occurs, before any yielding of the reinforcement, in a very brittle manner.

Increasing the gravity load and subsequent shear on the connection significantly reduces the drift capacity [7], [8], [10]. Lateral cyclic actions also reduces the stiffness and strength of the connection. [8], [10]. Punching shear failure due to seismic action is possible even if the acting shear on the slab critical section does not exceed the shear strength for monotonic loads. This is based on the assumptions that the shear capacity of the slab degrades and failure occurs when the shear capacity degrades to the point where it equals the shear demand [9], [11].

3. Behavior of slab-column connections under seismic actions

The overall seismic response of flat-slab buildings connections depends on the hysteretic properties of the slab-column connection. Punching shear failure for interior connections have been shown to be highly dependent of the gravity shear ratio $V_g/V_{rd,c}$ (where $V_g$ is the shear due to gravity loads without moment about the slab critical section) as reported by Pan and Moehle (1989) [7] Robertson and Durrani (1992) [8], Moehle (1996)-31 [9] and Ramos et al. (2014-13)-13 [10]. Parameters such as the gravity shear ratio and the lateral inter-story drift should be controlled to ensure that the slab-column connections maintain its capacity to transfer vertical loads under seismic actions.

Figure 2. Plots of load-rotation curves for tests by Kinnunnen and Nylander [6]

Figure 3. Shear strength degradation [11]

3.1. Slab-column connections with shear reinforcement

The addition of shear reinforcement has been shown to increase the deformation capacity (and thus ductility) and strength. [12]. High drift capacities can be obtained if shear reinforcement is considered. [10], [11], [13], [14]. In particular, substantially larger drifts capacities can be achieved when headed studs are adopted for the punching shear reinforcing system. Recently, Ramos et al., (2014) [10] showed that drift capacity for reinforced concrete slabs without transverse reinforcement may be improved four to five times when shear reinforcement in form of headed shear studs is adopted. Figure 3 refers to a slab without shear reinforcement and Figure 4 and 5 to slabs...
with reinforcing systems arranged radially and cruciformly, respectively. As shown in Figures 4 and 5, the drift capacity of the connection is increased 4 to 5 times.

Figure 4. Hysteretic diagram: slab without shear reinforcement \((\frac{V_g}{V_{Rd,c}} = 0.5)\) [10]

Figure 5. Hysteretic diagram: slab with shear reinforcement arranged radially from the column \((\frac{V_g}{V_{Rd,c}} = 0.5)\) [10]

3.2. Shear reinforcement systems, layouts and recommendations

There are several types of shear reinforcement. The most common and used systems are shear headed studs and stirrups. The efficiency of types of shear reinforcement is strongly influenced by their arrangement (layout) and detailing rules.

Nonetheless, the current Eurocodes (2 and 8) do not acknowledge such differences and thus the same design formulas are used for all systems. ACI 318-14 [15] seems to be more developed in this topic, presenting different design rules for stirrups and studs and ACI 352.1R-11 for shearbands (prebent steel strips). Design layouts are also provided for the different systems. For stirrups, effective anchorage is given when vertical leg is hooked around a longitudinal reinforcing bar [11] and for headed studs is provided by a head with an area equal to 10 times the cross sectional area of the stud. A recent experimental campaign by Cheng et al. (2008) [16] show that relatively large spacing occurring at the corners of rectangular columns may reduce the efficiency of the headed studs, because the highest shear stress occurs in the columns corners (square and rectangular columns) [17].

Moreover, the dynamic tests by Kang and Wallace (2006) [14] have shown the importance of reinforcing the slab-column interface, where the shear strength degradation is relevant. The first line of shear reinforcement should not be too close to the face of column nor too far, otherwise its effectiveness is very low. At each side of the column the distance \(a\), defined on Figure 5,
shall not be higher than 2d, according to ACI 318-14 (or 1.5d according to EC2 -1).

Figure 7. Layout of shear reinforcement: recommended layout (left) and not desired layout (right)

3.3. Recommended lateral drifts limits

The actual EC 8-1 defines a damage limitation requirement (DLR) in which inter-story drift shall be limited. The worst case scenario is for buildings having non-structural elements of brittle materials attached to the structure, where the inter-story drift must be limited do 0.5% for serviceability conditions (ELS). The Portuguese National Annex defines the reduction factor ν as 0.40 for seismic actions type 1. The basis of this DLR is to avoid limitation of use with high costs and withstand a more frequent seismic action without damage.

Experimental results indicate that the inter-story drift ratio should not exceed 1.5% for ultimate limit state (ULS) and the ductility can be ensured if \( V_g/V_{rd.c} < 0.4 \) as concluded by Pan and Moehle [7].

For seismic design, ACI 318-14 permits to use a design ductility criteria based on experiments that give an empirical relationship between the parameter \( V_g/V_{rd.c} \) and the drift ratio capacity of the structure. This criterion focuses on the ductility of the structures and, thus, the ductility of the slab-column connection and indicates if shear reinforcement is required for a given \( V_g/V_{rd.c} \) and drift capacity. This empirical relationship is based on research (Megally and Ghali (2002) [18] and Moehle (1996) [9] and it was initially introduced on ACI 318-05. The criterion is defined on equation 5 and plotted on figure 6

\[
\frac{\Delta_x}{h_{xx}} \geq 0.035 - \left( \frac{1}{20} \right) \left( \frac{V_g}{V_{rd.c}} \right)^{\frac{1}{2}}, \quad \text{mas} \quad \frac{\Delta_x}{h_{xx}} \leq 0.005 \quad (5)
\]

The requirement can be satisfied by adding shear reinforcement. According to ACI 352.1R-11, a minimum amount of shear reinforcement shall be provided:

\[
V_c \geq 0.29 \sqrt{f_{cd}} b d \quad [MPa] \quad (6)
\]

A comparison of the damage limitation requirement (EC 8-1) and the maximum drift for a given \( V_g/V_{rd.c} \) is presented on Figure 9. It is possible to conclude that if the damage limitation requirement defined on EC8 -1 is respected and \( V_g/V_{rd.c} < 0.45 \) then, according to Eq.5 criterion, the slab-column connection has enough ductility. Thus, the maximum drift ratio of the structure for ULS is 1.25% for \( V_g/V_{rd.c} = 0.45 \), according to Figure 9.

Even though there is not any direct relationship of those two drift limit recommendation, once the EC8-1 limit is related to the reduction of economic losses and ACI 318-14 is related with punching shear design, it is possible that the criterion given by EC8-1 might be a good indicator of the seismic behavior of slab-column connection.
4. Cases Studies description

The analyzed structure for the first case study (CE_1) is a 10 floor building with 2 basements, located in seismic zone 1.2 and founded in a class B soil. The slab thickness is 0.22 m and the capital thickness is 0.35m. The materials used in the case study are concrete C35/45 and the steel A500 NR SD. The gravity loads considered are 2.5 kN/m² for dead loads and 4.0 kN/m² for live loads, on all floors. Figure 10 summarizes the general characteristics of the analyzing structure. The behavior coefficients adopted are 3.6 for X direction and 3.0 for Y direction.

The second case study (CE_2(A)) is a building with the same plan dimension of the previous case study described but with just 5 floors and the vertical elements were design for the following criterion: \( \sigma_e \leq 0.8 \cdot f_{cd} \) for fundamental combination. All vertical elements were selected with a rectangular cross section of 0.30x0.50 m². The behavior coefficient adopted is 3.6 for both directions. The concrete used is C30/37.

Figure 10. Plan dimensions of the analyzing of the first study case

The second case study (CE_2(A)) is a building with the same plan dimension of the previous case study described but with just 5 floors and the vertical elements were design for the following criterion: \( \sigma_e \leq 0.8 \cdot f_{cd} \) for fundamental combination. All vertical elements were selected with a rectangular cross section of 0.30x0.50 m². The behavior coefficient adopted is 3.6 for both directions. The concrete used is C30/37.

Figure 11. Plan dimensions of the analyzing of the second study case

4.1. Structural analysis

Two methods to assess the effects of the seismic actions are used. On the first one (M1), the model for analysis of the effect of the seismic action comprises all slab-columns connections for a lateral force resisting system. On the second one (M2), the model consists on the exterior vertical elements, once plastic hinges are introduced on top and bottom of which central column to simulate a null flexural stiffness.

The CE_1 respected the imposed limits given by EC8-1 for the total contribution to lateral stiffness of all secondary seismic members (central columns). The table 1 shows the contribution of the central columns for total lateral stiffness and the relation \( \Delta \) between the contribution of the central columns and the contribution of all primary seismic members. The requirement is checked once \( \Delta \leq 15\% \). The DLR is also checked for both directions.
Figure 12 shows the relationship drift-height for ULS and ELS ($\nu = 0.4$) applying M1. The dashed red line indicates the drift limit of 0.5% given by EC8-1 for the DLR.

Table 1. Contribution of central columns for lateral stiffness

<table>
<thead>
<tr>
<th>Direction</th>
<th>Central columns</th>
<th>Relation, $\Delta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>X</td>
<td>2.6%</td>
<td>5.5%</td>
</tr>
<tr>
<td>Y</td>
<td>1.5%</td>
<td>2.4%</td>
</tr>
</tbody>
</table>

5. Results

In this section the design loads and the shear strength applying EC2-1 and MC2010 for each study case are presented (only for Y direction). A comparison with the results obtained by applying the suggested methodology on EC8-1 based on primary and secondary members is done. The design values are obtained using a modal response spectrum analysis.

Table 2. Contribution of central columns for lateral stiffness

<table>
<thead>
<tr>
<th>Direction</th>
<th>Central columns</th>
<th>Relation, $\Delta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>X</td>
<td>26.2%</td>
<td>35.5%</td>
</tr>
<tr>
<td>Y</td>
<td>29.8%</td>
<td>42.4%</td>
</tr>
</tbody>
</table>

5.1. Case Study 1

Table 3 shows the design loads for CE_1 according to EC2-1. Applying MC 2010, the design punching shear is not affect or majored to take in account the eccentricity due the moment $\Delta M_{Ed}$. Thus, the design values are the same assessed by applying method M2.

Table 3. Design moments and shear on the connection according to EC2-1

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>$m_{Ed}$ [kN.m/m]</th>
<th>$V_{Ed}$ [kN]</th>
<th>$m_{Ed}$ [kN.m/m]</th>
<th>$V_{Ed}$ [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Persistent</td>
<td>188</td>
<td>844</td>
<td>188</td>
<td>797</td>
</tr>
<tr>
<td>Seismic (Y)</td>
<td>199</td>
<td>570</td>
<td>76</td>
<td>373</td>
</tr>
</tbody>
</table>

Slab flexural reinforcement on the capitals is $\varnothing 12/\varnothing 20+\varnothing 16/\varnothing 20$ ($\rho = 0.0051\%$). For this reinforcement ratio, $m_{Ed} = 200$ kN. Table 4 presents the shear strength for both structural codes for method M1.

For seismic design following the criterion given by Eq.(5), the design story drift and the gravity shear ratio must be known. Applying this criterion with
EC2 considerations, \( V_g \) is the shear due the component \( g + \psi_2 \cdot q \) of the seismic combination.

Table 4. Shear strength according to EC2 and MC 2010

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>( V_{rd,c} ) [kN]</th>
<th>( V_{rd,c} ) [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Persistent</td>
<td>1136</td>
<td>1126</td>
</tr>
<tr>
<td>Seismic (Y)</td>
<td>1136</td>
<td>736</td>
</tr>
</tbody>
</table>

The design drift and the gravity shear ratio for M1 method are presented on Table 5. Figure 14 illustrates the criterion given by Eq.(5) for CE_1.

Table 5. Design drift and gravity shear ratio – M1

<table>
<thead>
<tr>
<th></th>
<th>EC 2-1</th>
<th>MC2010</th>
</tr>
</thead>
<tbody>
<tr>
<td>( V_g )</td>
<td>398</td>
<td>373</td>
</tr>
<tr>
<td>( V_g/V_{rd,c} )</td>
<td>0,35</td>
<td>0,18</td>
</tr>
<tr>
<td>( d_r/h )</td>
<td>0,82%</td>
<td>0,82%</td>
</tr>
</tbody>
</table>

Figure 14. Illustration of Eq.(5) criterion for method M1

Once the lateral stiffness of the structure is very high, due to the shear walls, the difference of the design drift ratios evaluated by method M1 and M2 is not relevant. For M1 the design drift is 0,82% as shown on Table 5, and for M2 is 0,87%.

Applying the EC8-1 philosophy for the design of the secondary seismic members, the design values such as \( \Delta M_{Ed} \) are obtained using a modal response spectrum analysis with a behavior coefficient of 1,0, since slab-column connection must maintain in the elastic range, which means that yielding can not occur. The value of \( \Delta M_{Ed} \) must be majored by a factor, which is given by the ratio between the displacement on the top of building with and without the secondary members considered on the model. This is used to correct the internal forces in secondary members obtained in a global model and the design displacements are obtained in a model using just the primary seismic members.

Table 6 shows the design value for moment and shear on the slab-column connection and the shear strength given by EC2-1.

Table 6. Design values for EC8-1 method

<table>
<thead>
<tr>
<th></th>
<th>( m_{Ed,lab} ) [kN.m/m]</th>
<th>( \Delta M_{Ed,x} ) [kN.m]</th>
<th>( \beta )</th>
<th>( V_{Ed} ) [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>394</td>
<td>569</td>
<td>2,52</td>
<td>953</td>
</tr>
</tbody>
</table>

For those design values, a flexural reinforcement ratio of 1,1% is obtained which represent an increase to double the amount of reinforcement. Comparing both designs, it is possible to conclude that deformation capacity of the slab-column connection based on the previous method to assess the design forces and displacements is higher according to Figure 2.

5.2. Case Study 2(A)

For CE_2(A) the values of design drifts (ULS) obtained are 1,75% for method M1 and 2,07% for method M2. The design punching shear and shear strength for method M1 are presented on Table 7.

Table 7. Design shear assessed by method M1 and shear strength

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>( V_{Ed} ) [kN]</th>
<th>( V_{rd,c} ) [kN]</th>
<th>( V_{Ed} ) [kN]</th>
<th>( V_{Ed} ) [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Persistent</td>
<td>814</td>
<td>961</td>
<td>766</td>
<td>912</td>
</tr>
<tr>
<td>Seismic (Y)</td>
<td>747</td>
<td>961</td>
<td>408</td>
<td>369</td>
</tr>
</tbody>
</table>
The design drift and the gravity shear ratio for M1 method are presented on Table 5 and the values are plotted on Figure 15.

Table 8. Design drift and gravity shear ratio – M1

<table>
<thead>
<tr>
<th></th>
<th>EC 2-1</th>
<th>MC2010</th>
</tr>
</thead>
<tbody>
<tr>
<td>$V_g$</td>
<td>434</td>
<td>408</td>
</tr>
<tr>
<td>$V_g/V_{Rd,c}$</td>
<td>0,51</td>
<td>0,36</td>
</tr>
<tr>
<td>$d_r/h$</td>
<td>1,75%</td>
<td>1,75%</td>
</tr>
</tbody>
</table>

According to EC2-1 punching shear is checked. However, the ACI 318-14 criterion for design punching shear is not satisfied. Even though the slab shear strength is enough, the connection does not have the desired ductility for the seismic action, according to ACI318-14 criterion. This conclusion was expected since the DLR is not verified as shown on section 4.

5.3. Study Case 2(B)

One way to decrease inter-story drift, moment and the shear on the slab column connection is to increase the lateral stiffness of the structure. Vertical elements on the edge of the building were modified. The columns on X direction have cross sectional area of 0,85x0,30 m² and on Y direction 1,10x0,30 m². Corner columns maintains their dimensions. The criterion to redesign vertical elements was limiting the overall lateral contribution of central column to 15% on each direction. For this situation the value of the inter-story drift for ELS is 0,51% (method M1) and 0,55% (method M2). The values obtained are significantly different from those of CE_2(A), showing the importance of the lateral stiffness of the structure. However, the design drift applying the method M1 is 1,27% and $V_g/V_{Rd,c} = 0,47 (>0,45$, thus the Eq.(5) criterion is not satisfied because the drift limit is 1,13%. In this way, shear reinforcement must be adopted to enhance the slab-column connection behavior, in particular the ductility. It is desired that high deflections develop prior to punching failure rather than a brittle failure, in order to advice for potential punching failure.

6. Conclusions

- Slab-column connections subjected to lateral cyclic loads experience very complex behavior that is not properly studied and yet not well documented.
- The punching shear capacity is influenced by the flexural reinforcement ratio. Increasing the reinforcement causes higher punching shear strength but strongly reduces the ductility of the slab and therefore reduces the deformation capacity.
- One possible way to increase the slab-column connection’s strength, but mainly the ductility, is to use punching shear reinforcement
- The EC8-1 damage limitation requirement impose a drift limit for serviceability conditions in order to ensure a minimum lateral stiffness and withstand a more frequent seismic action without damage. Comparing it to the ACI 318-14 ductility criterion, the EC8-1 damage limitation seems to be a good indicator of the ductility and therefore a good performance indicator of slab-column connections behavior subjected do seismic actions.
Current codes, such as Eurocode 2, do not acknowledge the different shear reinforcement systems and therefore, same design formulas or detailing rules are adopted for all systems.

References


[15] ACI Committee 318 (2014), Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14),

