NUMERICAL INVESTIGATION ON PUNCHING SHEAR OF SLAB-COLUMN CONNECTIONS SUBJECTED TO SEISMIC LOADING

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ABSTRACT

In North American practice, the design of flat slabs under seismic loading is covered by the largely empirical requirements of ACI 318-14 as well as the more detailed provisions of ACI 352.1R-11. However, the situation is rather different in European practice where the current seismic code (EN 1998) does not provide any guidance on the design of flat slabs. This study attempts to bridge the gap between the empirical design methods of North America, nonlinear finite element analysis (NLFEA) and the mechanically based critical shear crack theory (CSCT). It is proposed that the current CSCT failure criterion may still be used in seismic conditions so long as an appropriate choice of slab rotation is made. This hypothesis is evaluated by assessing five internal slab-column connections from the literature that were subjected to cyclic loading. The slabs are also assessed with NLFEA. It is shown that the CSCT gives reasonable results when the shear resistance is related to the greatest of the relative slab-column rotation and the rotation under gravity load alone instead of the maximum slab rotation suggested in fib Model Code 2010. Use of the relative slab-column rotation is beneficial for design purposes since it can be simply related to lateral drift which is a limiting parameter in seismic design. Thus, it is suggested that the CSCT could form the basis of future design recommendations for seismically-loaded flat slabs.

Keywords: punching shear; flat slabs; seismic provisions; the critical shear crack theory; nonlinear finite element analysis

1. INTRODUCTION

Flat slabs are commonly used in building structures because of their practical advantages, including: short construction time; flat soffit; and flexibility in column positioning. Although this structural system is commonly used in regions of low to moderate seismic risk, the use of flat slabs is prohibited in high seismicity regions due to limited ductility and energy dissipation capacity, primarily caused by brittle punching failure. In the 1985 Mexico City earthquake, it was reported that at least 91 flat slab buildings collapsed due to punching failure (Ghali and Megally, 2000). More recently, most flat slab buildings throughout southern California were severely damaged in the 1994 Northridge earthquake (Hueste and Wight, 1999). On the basis of these disastrous events, seismic building codes typically only permit flat slabs to be used as gravity-load-carrying systems with special moment frames or special structural walls used as the primary lateral-force-resisting system (LFRS). In this case, the lateral deformation of the flat slab system is controlled by that of the primary LFRS. Thus, it is important to ensure that the flat slab system can undergo this deformation without incurring so-called drift-induced punching failure. Although numerous previous investigations have studied the behaviour of flat slab-column connections, there is still a lack of understanding of the influence of cyclic loading history on punching resistance. This can be attributed to the observation that almost no comparative tests were carried out on pairs of identical slabs subjected to both monotonic and cyclic loading prior to those of Drakatos et al. (2016)

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who tested five pairs of slabs. The current study shows numerically that cyclic loading leads to an accumulation of plastic rebar strain which increases crack opening with successive loading cycles. Thus, both the unbalanced moment capacity and ductility of cyclically-loaded specimens degrade faster than in identical monotonically loaded specimens.

The paper also briefly reviews current North American and European seismic provisions for the design of slab-column connections. Previous studies which have significantly influenced these seismic provisions are discussed along with the Critical Shear Crack Theory (CSCT) of Muttoni (2008). Results acquired from nonlinear finite element analysis (NLFEA) of slab specimens tested by Drakatos et al. (2016) and Tian et al. (2008) are presented and discussed. Lastly, the capability of CSCT to predict punching failure under seismic loading is evaluated against experimental results from five internal slab-column connections, without shear reinforcement, tested by Drakatos et al (2016). The accuracy of the CSCT predictions is shown to be reasonable in comparison with commonly used empirically-based models.

2. LITERATURE REVIEW

2.1 Previous studies on seismic behaviour of slab-column connections

As stated earlier, seismically loaded flat slab buildings are required to have sufficient ductility to attain the design drift limit without losing their capability to sustain gravity loads. Previous studies by Pan and Moehle (1989), Robertson and Durrani (1993), Megally and Ghali (1994), and Hueste and Wight (1999) found the ductility of slab column connections to be mainly affected by the gravity shear ratio (GSR) which is defined as the ratio of shear stress induced by gravity load to the shear resistance provided by concrete alone. The ductility of slab to column connections decreases gradually with increasing GSR. It is common in North American practice to limit the GSR to below 0.4 in order to ensure that connections can achieve 1.5% lateral drift prior to punching. Another important parameter found to influence the behaviour of slab-column connections is reversed-cyclic loading history. A study by Megally and Ghali (1994) reported that the shear resistance provided by concrete is less under reversed-cyclic than monotonic loading. This phenomenon can be explained with reference to study into aggregate interlock conducted by Walraven (1980). It was found through observations of push-off tests that the contact area between crack surfaces reduces under reversed-cyclic loading. Consequently, aggregate interlock, which is primarily influenced by the contact surface area, deteriorates faster under cyclic loading. The recent tests of Drakatos et al. (2016) show that the connection behaviour is also influenced by the amount of flexural reinforcement adjacent to the slab-column connection. Drakatos et al. concluded that slab-column connections with lower flexural reinforcement ratios are most susceptible to degradation under cyclic loading.

2.2 Seismic provisions for slab-column connections in high seismicity regions

In North American practice, the design of flat slabs for seismic loading is covered by the largely empirical requirements of ACI 318-14, as well as the more detailed guidance of ACI 352.1R-11. Section 18.14.5.1 of ACI 318-14 requires shear reinforcement satisfying the requirements of section 8.7.6 or 8.7.7 to be provided in any slab critical section defined in section 22.6.4.1 if

\[
\frac{\Delta_s}{h_{ss}} \geq 0.035 - \left( \frac{1}{20} \right) \left( \frac{\nu_{ug}}{\phi \nu_e} \right)
\]

(1)

where \(\frac{\Delta_s}{h_{ss}}\) is the largest inter-storey drift in critical floors, \(\nu_{ug}\) is the factored shear stress on the slab critical section for two-way action due to gravity loads without moment transfer, \(\nu_e\) is the two-way shear strength provided by concrete and \(\phi\) is the capacity reduction factor for shear. ACI 352. 1R-11 requires the shear resistance provided by concrete \(\nu_e\) to be reduced by 25% for slab-column connections subject to significant flexural yielding prior to failure.

By contrast, Eurocode 8 (BS EN 1998-1:2004) provides no specific guidance on the design of slab-column connections in regions of high seismicity. Section 4.2.2 of the code briefly mentions that a
certain number of structural members may be designated as “secondary” seismic members (or elements), not forming part of the seismic action resisting system of the building. Secondary seismic members are defined as structural members which contribute less than 15% of the lateral stiffness provided by all primary seismic members. By this definition, most flat slab systems should be treated as secondary seismic members. Additionally, Eurocode 8 neglects the contribution of secondary elements to the global strength and stiffness of the structure. Nonetheless, these members and their connections are required to be designed and detailed to maintain support of gravity loading when subjected to the displacements imposed by the most unfavourable seismic design condition. This requirement is the same as that of the North American provisions but no guidance is provided to assist designers in achieving this goal.

2.3 A brief review of the Critical Shear Crack Theory (Muttoni, 2008)

The so-called the Critical Shear Crack Theory proposed by Muttoni (2008) is a mechanically-based model for calculating punching resistance of two-way slabs. The theory assumes that shear resistance depends on the width of the so called critical shear crack, which in turn depends on the slab rotation (ψ) and slab effective depth (d) (Figure 1a). According to the theory, a critical inclined crack develops diagonally from the column face disturbing the shear-transfer actions within the compressive strut region (Figure 1b) thereby eventually triggering punching failure.

Figure 1. (a) Correlation between opening of critical shear crack, thickness of slab, and rotation ψ; (b) inclined compression strut carrying shear disturbed by the critical shear crack (Muttoni, 2008)

For concentric loading at internal slab-column connections, the shear resistance provided by aggregate interlock along the critical crack is determined as follows:

\[
\frac{V_R}{b_0d/f'_c} = \frac{0.75 \delta_t}{1 + 15 \frac{\psi d}{d_0 + d_g}} \quad (SI \ units; N, \ mm) \tag{2}
\]

where \(V_R\) is the punching load capacity, \(b_0\) is the perimeter of the critical section at \(d/2\) from the column face, \(\psi\) is the slab rotation, \(d_g\) is the maximum aggregate size, and \(d_0\) is a reference aggregate size equal to 16 mm. As can be seen from Equation 2, the shear resistance is inversely related to a single value of slab rotation (\(\psi\)). For concentric loading, the slab rotation is almost uniform along the tangential axis, but this is not the case for flat slabs subjected to seismic loading. The slab is divided into sector elements which represent parts of the slab between radial cracks (Kinnunen and Nylander, 1960). Results from the tests of Drakatos et al. (2016) and NLFEA results presented in this paper show that the slab rotation of the sector elements under seismic loading follows an approximately sinusoidal law, with the maximum rotation at the hogging face. Consequently, the shear resistance calculated with Equation 2 varies along the control perimeter according to the variation in slab rotation. The influence of choosing different values of slab rotation to the CSCT prediction will be assessed in detail later. Model Code 2010 (2013) reduces the length of the basic control perimeter by a coefficient of eccentricity \(k_e\) to account for the non-uniform shear force distribution induced by moment transfer between the slab and the supported area. The coefficient \(k_e\) can be determined as:

\[
k_e = \frac{1}{1 + e_u/b_u} \quad (SI \ units; N, \ mm) \tag{3}
\]
where $e_a$ is the eccentricity of the resultant of shear force with respect to the centroid of the basic control perimeter and $b_a$ is the diameter of a circle with the same surface as the region inside the basic control perimeter.

3. NONLINEAR FINITE ELEMENT ANALYSIS

Before conducting any parametric studies, the numerical model was validated by analysing 10 full-scale internal slab-column connections without shear reinforcement recently tested by Drakatos et al. (2016). The specimens measured 3.0 x 3.0 x 0.25 m. Five specimens were tested under monotonic loading with the remainder tested under cyclic loading. Details of the experimental test setup, specimen configuration, boundary conditions, and loading procedure are reported in Drakatos et al. (2016).

3.1 Methodology

ATENA (Cervenka Consulting) was used to perform nonlinear finite element analysis (NLFEA) of all the Drakatos et al. (2016) slab specimens. ATENA models concrete (cementitious2) with a smeared crack approach and a fracture-plastic model which combines constitutive models for tensile (fracturing) and compressive (plastic) behaviour. The fracture model is based on the classical orthotropic smeared crack formulation and crack band model proposed by Bazant and Oh (1983). The Rankine tensile fracture criterion and exponential softening can be used with either a rotating or fixed crack model to model concrete cracking. In this study, a fully rotating-crack model was used since it was found to model the experimental response more accurately than the fixed-crack model which tends to produce overly stiff load-rotation responses. Plasticity for concrete under compression is controlled by the Menetrey-William failure surface (Menetrey and William, 1995) which is expressed in terms of three independent stress invariants: hydrostatic stress ($\xi$), deviatoric stress ($\rho$), and deviatoric polar angle ($\theta$). After concrete cracks, the compressive strength in the direction parallel to the cracks is reduced following the recommendations of the Modified Compression Field Theory (MCFT) of Vecchio and Collins (1986). Taking advantage of symmetry, only half of the specimen was modelled. Brick elements with linear order (8-noded) were used to model the concrete slab whereas all other experimental apparatus, including steel arms and loading plates were modelled using linear tetrahedral (4-noded) elements with elastic material type. Reinforcement bars were modelled as 1D 2-noded linear truss elements. The embedded reinforcement option was adopted and perfect bond was assumed between reinforcement bars and concrete. Cyclic reinforcement model based on Menegotto and Pinto (1973) was used to include the Bauschinger effect in which reinforcement yield strength decreases when the direction of strain is changed. Previous calibration studies showed that 10 brick elements through the slab thickness are sufficient to capture accurately the out-of-plane shear behaviour of the slabs. To increase modelling accuracy and computational efficiency the mesh was refined locally around the column as shown in Figure 2 which also shows the loading arrangement.

Figure 2. Geometry, boundary conditions, and meshing for slabs analysed using NLFEA
Static analysis with force-control was used for the monotonic test along with Arc-Length iteration whereas dynamic analysis with displacement-control (Newton-Raphson) was used for the cyclic test in order to avoid convergence issues. The convergence tolerance was set at 1% for displacement, residual, and absolute residual error and 0.1% for energy error. The same concrete constitutive parameters were used for monotonic and cyclic tests in this study, but, in reality, degradation of concrete strength is more severe under cyclic loading. Consequently, the cyclic predictions could possibly be improved further by modifying the concrete parameters. In ATENA, the only additional constitutive parameter for concrete under cyclic loading is the unloading factor (UF) which controls the crack closure stiffness. UF ranges between 0 and 1 with 0 for unloading to the origin (default value for backward compatibility) and 1 for unloading parallel to the initial elastic stiffness. A sensitivity study of cyclically loaded punching specimens showed that UF mostly influences the shape of the hysteresis curve with UF=0 giving the best fit to the actual hysteretic behaviour. All important parameters used for conducting the numerical analysis with ATENA are summarised in Table 1 below.

Table 1. Summary of material parameters and numerical input for FE model in ATENA

<table>
<thead>
<tr>
<th>No.</th>
<th>Parameter</th>
<th>Value/Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>Fracture energy</td>
<td>Based on Model Code 90</td>
</tr>
<tr>
<td>A2</td>
<td>Smeared crack model</td>
<td>Fully-rotating crack</td>
</tr>
<tr>
<td>A3</td>
<td>Critical compressive displacement</td>
<td>0.5 mm</td>
</tr>
<tr>
<td>A4</td>
<td>Limit of compressive strength reduction due to cracking (MCFT)</td>
<td>0.8fc’</td>
</tr>
<tr>
<td>A5</td>
<td>Eccentricity (defining the shape of the failure surface)</td>
<td>0.52</td>
</tr>
<tr>
<td>A6</td>
<td>Volume dilatation plastic factor</td>
<td>0</td>
</tr>
<tr>
<td>A7</td>
<td>Unloading factor for cyclic loading</td>
<td>0</td>
</tr>
</tbody>
</table>

**Reinforcement bar model**

| B1  | Stress-strain relationship                    | Bilinear                                 |
| B2  | Bond-slip model                               | Perfect bond                             |
| B3  | Cyclic behaviour                              | Menegotto and Pinto (1973)               |

**Loading procedure and convergence criteria**

| C1  | Loading procedure for monotonic tests         | Static (force-controlled)               |
| C2  | Iteration method for monotonic tests          | Arc-length method                       |
| C3  | Loading procedure for cyclic tests            | Dynamic (displacement-controlled)       |
| C4  | Iteration method for cyclic tests             | Newton-Raphson method                   |
| C5  | Convergence criteria for displacement, residual, and absolute residual error | 1%                                     |
| C6  | Convergence criteria for energy error         | 0.1%                                     |

**Mesh properties**

| D1  | Mesh size (finest)                           | 25 x 25 x 25 mm                         |
| D2  | Mesh element for concrete slab                | 8-noded hexahedral (linear)             |
| D3  | Mesh element for loading apparatus           | 4-noded tetrahedral (linear)            |
| D4  | Mesh element for reinforcement bar            | 2-noded truss element                   |

3.2 Validation of NLFEA results with experimental test data

For the sake of conciseness, only the results of selected slab analyses are reported here. The first parameter to be considered is the unbalanced moment versus relative slab-column rotation. The relative slab-column rotation, \( \psi_{sec} \), is defined as:

\[
\psi_{sec} = \frac{\psi_{max} - \psi_{min}}{2} - \psi_{col} 
\]  

(4)
where $\psi_{\text{max}}$ is the maximum slab rotation, $\psi_{\text{min}}$ is the minimum slab rotation and $\psi_{\text{col}}$ is the column rotation. In the numerical model, the column rotation, $\psi_{\text{col}}$, is zero since both the top and bottom column plates were modelled as fully restrained and rigid. $\psi_{\text{max}}$ and $\psi_{\text{min}}$ relate to hogging and sagging regions of the slab, respectively. Figure 3 compares the measured and calculated moment-rotation responses for both monotonic and cyclic loading.

![Image](image_url)

**Figure 3.** Comparison of measured and predicted unbalanced moment versus slab-column rotation (%) for: (a) monotonic tests; (b) cyclic tests by Drakatos et al. (2016)

From Figure 3, it can be seen that the NLFEA gives reasonable predictions of the measured response but tends to slightly overestimate the measured stiffness and punching resistance, especially for cyclically-loaded slabs. The NLFEA overestimates the energy dissipation capacity in the cyclic tests because pinching behaviour is not modelled realistically in the current constitutive concrete model. For cyclically-loaded slabs, the predicted unbalanced moment capacity reduces gradually subsequent to reaching its peak value, while the experimentally observed failure is more brittle in nature.

Further validation is done by comparing the variation of the local slab rotation (i.e. rotation at different sector elements) as function of angle measured from the axis of the unbalanced moment vector. Figure
Figure 4. Comparison of measured and predicted slab local rotations at varying angles for: (a) monotonic tests; (b) cyclic tests by Drakatos et al. (2016)

Figure 4 shows that the profiles of the measured and predicted rotations compare reasonably well. The results are also in agreement with the analytical model proposed by Drakatos (2016) which assumes that the slab rotation of sector elements subjected to lateral loading varies sinusoidally with regard to the axis of applied unbalanced moment.

Figure 5. Comparison of observed (experimental) and predicted (NLFEA) crack patterns at failure for: (a) PD3 (monotonic); (b) PD8 (cyclic) adapted from Drakatos et al. (2016)
Figure 5 shows that the measured and predicted crack patterns, including crack angle, compare well at failure. It is concluded that the adopted NLFEA procedure produces reasonably accurate results making it suitable for conducting further parametric studies.

4. PARAMETRIC STUDIES

Parametric studies were carried out using the same numerical approach but for a different experimental test setup. Specimen L0.5 of Tian et al. (2008) was chosen for these parametric studies since the test arrangement is representative of that typically used in North American studies. Details of the authors’ analysis of specimen L0.5, including reinforcement strain and maximum slab rotation, can be found in Setiawan et al. (2017). The parametric studies consider the influence of: a) number of loading cycles (n); b) GSR and c) percentage of hogging flexural reinforcement ratio (rho) in the column strip. Three different numbers of loading cycles (n=1, 3, or 5) were applied to determine the influence of repetitive loading on the slab-column connection behaviour. The GSR was varied between 0.28 and 0.55 while the hogging reinforcement ratio in the column strip was varied between 0.3% and 1.75%. For each variation of GSR and hogging reinforcement ratio, separate monotonic and cyclic loading analyses were undertaken in order to determine the influence of cyclic degradation.

The influence of varying the number of loading cycles can be seen in the hysteresis and backbone responses of the slab-column connections shown in Figure 6 below.

![Hysteresis response](image1)

![Backbone curve](image2)

Figure 6. Influence of different number of loading cycle: (a) hysteresis response; (b) backbone curve for slabs specimens with GSR=0.44 and rho=0.5%

Figure 6 shows that slab specimens loaded with more loading cycles experience earlier punching failure, indicated by significant strength and stiffness degradation at lower drift level, compared to other specimens. This occurs because the strain in the reinforcement bars increases progressively with the number of loading cycles. This accumulation of plastic rebar strain leads to an increase in crack opening for successive cycles as reported by Drakatos et al. (2016). In the tested specimens, the shear resistance provided by aggregate interlock degrades faster under cyclic than monotonic loading. However, degradation of shear resistance along cracks is not modelled in the ATENA analyses since a rotating crack model was used. Despite this, the physical effect of loss of aggregate interlock is modelled indirectly in ATENA, since concrete damage is controlled by the accumulated equivalent plastic strain. Accumulation of strain in the inelastic phase causes concrete to enter the compression softening regime more rapidly under cyclic than monotonic loading. Damage is also enhanced under cyclic loading since the compressive response is adversely affected by damage in tension and vice versa.
Figure 7. Backbone curve of slab specimens with: (a) varying gravity shear ratio; (b) varying hogging reinforcement ratio

Figure 7a shows that the peak unbalanced moment reduces with increasing GSR as observed in many previous experimental studies. Figure 7b suggests that increasing the hogging reinforcement ratio increases the maximum unbalanced moment capacity but reduces the connection ductility. Since ductility is critical for seismic design, it is suggested that consideration should be given to relating the lateral drift limit to the flexural reinforcement ratio in addition to the GSR currently considered in ACI 318-14. As mentioned earlier, each of the specimens shown in Figure 7a and 7b was analysed under both monotonic and cyclic loading in order to isolate the influence of cyclic loading. The influence of cyclic degradation on peak unbalanced moment and failure drift are shown in Figure 8 and 9 respectively as functions of GSR and hogging reinforcement ratio.

Figure 8. Influence of cyclic degradation to: (a) peak unbalanced moment; (b) failure drift as a function of GSR (with constant rho=0.5%)

Figure 9. Influence of cyclic degradation to: (a) peak unbalanced moment; (b) failure drift as a function of rho (with constant GSR=0.44)
Figure 8a demonstrates that the slab-column connection loaded with the lowest GSR experiences the greatest reduction in peak unbalanced moment capacity under cyclic loading, as found experimentally by Drakatos et al. (2016). Figure 8b shows that cyclic loading reduces the drift at failure but the degree of degradation is scattered. Furthermore, Figure 9a shows that peak unbalanced moment for specimens with lower hogging reinforcement ratios is less affected by cyclic degradation since their flexural strength limit is reached prior to failure. It can be seen from Figure 9b that the hogging reinforcement ratio also influences the degree of cyclic degradation. The specimen with the lowest reinforcement ratio (r=0.3%) experiences the greatest reduction of drift at failure. Generally, it can be concluded that cyclic loading causes a reduction in both the peak unbalanced moment and the drift at failure. The reduction is influenced by the GSR and hogging reinforcement ratio.

5. IMPLEMENTATION OF THE CRITICAL SHEAR CRACK THEORY FOR SEISMIC CASE

This section investigates the application of the CSCT to five seismically-loaded slabs tested by Drakatos et al. (2016). The CSCT failure criterion was assessed using cyclic slab moment-rotations calculated with ATENA as well as the measured rotations from experimental test. As stated earlier, the CSCT failure prediction depends on the choice of rotation input into Equation (3) as well as k, which accounts for eccentricity. The choice of rotation for use in the calculation of punching resistance is unclear since shear failure does not occur when the failure criterion is first reached in the sector with maximum rotation because of the presence of shear redistribution (Sagaseta et al., 2011). To investigate this phenomenon further, this study compares the shear resistance calculated with both the maximum slab rotation (ψ_max) and the greatest of the initial slab rotation due to gravity load alone (ψ_grav) and the relative slab-column connection rotation (ψ_scc) from Equation 4. The gravity load rotation ψ_grav governs when eccentricity is low (stress-induced punching) since ψ_scc for pure gravity loading is zero. The effect of non-uniform distribution of shear forces due to eccentricity was accounted for with k, from Equation 3. According to the CSCT, failure occurs when the calculated shear resistance (k, V_k) at rotation ψ equals the applied gravity shear force. Since the main issue in this study is the consideration of seismic loading, the variable assessed here is only the failure drift (deformation limit). The drift limit predicted by CSCT is plotted along with the prediction of various empirically-based models in Figure 10.

![Drift predictions with load-rotation response from experimental test](image1)

(a) Drift predictions with load-rotation response from experimental test

![Drift predictions with load-rotation response from ATENA (NLFEA)](image2)

(b) Drift predictions with load-rotation response from ATENA

Figure 10. Prediction of drift limit according to CSCT, ACI 318-14, Megally and Ghali (1994), and Hueste and Wight (1999) with cyclic load-rotation response acquired from: a) experimental test; b) ATENA

Figure 10a and 10b show that the best CSCT predictions of ultimate drift are obtained using the relative slab-column rotation (ψ_scc) (which exceeded ψ_grav), with use of the maximum rotation tending to significantly underestimate the drift limit. Use of ψ_scc is advantageous for assessment since the relative slab-column rotation can easily be related to the design lateral drift limit. Further assessment of a larger slab database is underway to validate this finding. Comparison of Figure 10a and 10b shows that the drift limit is best predicted by the CSCT when the measured rather than calculated load-rotation response
(ATENA) is used. Despite this, the ultimate drift limit prediction obtained from the ATENA relative slab-column rotation is reasonably accurate although rather conservative at the lowest considered GSR. Of the empirically-based models, ACI 318-14 produces relatively safe results (except for the slabs with largest GSR) while Megally and Ghali (1994) and Hueste and Wight (1999) tends to overestimate the drift limit. Of all the predictions, only the CSCT ones are derived from a mechanically based model. This confirms the proposal of Drakatos et al. (2016) that the CSCT forms a rational basis of future design recommendations for seismically-loaded flat slabs.

6. CONCLUSION

This paper uses NLFEA with ATENA to investigate the influence of seismic loading (cyclic) on the punching resistance of slab-column connections without shear reinforcement. The influence of number of loading cycles, GSR and hogging reinforcement ratio on the degree of cyclic degradation was studied through parametric studies. The application of the CSCT to seismically loaded slabs was also assessed by evaluating five internal slab-column connections subjected to reversed-cyclic loading. The main conclusions of the paper are:

1. Nonlinear finite element analysis with ATENA was shown to produce reasonably accurate predictions of the behaviour of slab-column connections subjected to reversed-cyclic loading. NLFEA with solid elements is not only capable on predicting the global moment-rotation response but also predicting more detailed aspects such as sector (local) slab rotations and crack patterns.
2. Comparisons of the response of slab-column connections loaded with different numbers of loading cycle, suggests that repetitive (cyclic) loading leads to a premature punching failure due to the accumulation of plastic rebar strains. These unrecovered plastic strains cause cracks in concrete to open wider for successive cycles. Consequently, the capability of aggregate interlock to transmit the shear forces along these cracks deteriorates faster than for monotonic loading. It is interesting to note that NLFEA was able to reproduce this degradation with reasonable accuracy.
3. The experimental results of Drakatos et al. (2016) and NLFEA results from this study show that the degree of cyclic degradation is influenced by the GSR and hogging reinforcement ratio. Slab-column connections with lower GSR or hogging reinforcement ratios are most susceptible to cyclic degradation. It is suggested that the current design formula (Equation 1) in ACI 318-14 for the limiting drift is updated to include the influence of the hogging reinforcement ratio as well as GSR.
4. Comparing different approaches on predicting the failure deformation of slabs from the database, it was found that CSCT and ACI 318-14 produce safe and reasonably accurate predictions, whereas the empirically drift-models proposed by Megally and Ghali (1994) and Hueste and Wight (1999) slightly overestimate ductility. The CSCT appears to produce the best predictions of ultimate drift when the relative slab-column rotation is used. For design purposes, use of the relative slab column rotation is beneficial since it can be easily related to the lateral drift which is generally used as the main seismic design consideration.

7. ACKNOWLEDGEMENTS

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8. REFERENCES

ACI (American Concrete Institute) (2014) ACI Committee 318: Building code requirements for structural concrete and commentary. American Concrete Institute, Farmington Hills, MI, USA.

ACI 352.1R-11 (2012). Guide for Design of Slab-column Connections in Monolithic Concrete Structures, American Concrete Institute.


