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THE EFFECT OF SLAB AND TRANSVERSE BEAMS ON THE BEHAVIOUR OF FULL-SCALE PRE-1970'S RC BEAM-COLUMN JOINTS

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Abstract

Past research and empirical evidence has shown that beam-column joints play a critical role in the cyclic behaviour of reinforced concrete (RC) structures and that a good understanding of the complex mechanical interactions in beam-column connections is necessary to ensure the desired hierarchy of failure is achieved. Development of such understanding requires specifically designed numerical and experimental studies that depict realistic structures. However, reviews of published experimental literature show that most experimental studies on the seismic behaviour pre-1970's RC beam-column connections focus on the testing of sub-assemblies without slabs or transverse beams, which are unrepresentative of reality.

This paper presents the first part of a study being carried out to shed light on how (and by how much) slabs and transverse beams contribute to the seismic performance of gravity-designed interior beam-column connections, and the consequences of ignoring or wrongly representing this contribution. It presents the results of experiments on three full-scale beam-column joints. One of these test specimens is designed to Eurocode 8 and two are of typical pre-1970's design, with one specimen containing a slab and transverse beams, whilst the other represents the typical cross-shaped specimen tested in many other studies. The paper also presents preliminary nonlinear three-dimensional finite element (FE) models to complement the experiments, and extend them to look at the case of a pre-1970's specimen with transverse beams and no slab. As expected, the experimental and numerical results clearly demonstrate that the progression of damage and failure mechanisms in beam-column connections with and without slabs and transverse beams differ significantly. The experimental findings also generally confirm inferences from the limited published experimental evidence that the contribution of slabs and transverse beams can be underestimated by current guidelines. These observations have implications when considering common simplifications made in the numerical modelling of RC moment resisting frames when assessing their seismic performance.

Keywords: beam-column joints, slabs, transverse beams, reinforced concrete, seismic behaviour

1. Introduction

Recent earthquakes have highlighted the poor seismic performance of non-seismically detailed reinforced concrete (RC) moment resisting frames (MRF) built before the 1970's. Global failure of these structures are typified by the formation of weak column – strong beam mechanisms, leading to soft-storey failures. It is evident that in order to reduce the economic and life-losses associated with such buildings they should either be strengthened or replaced. In the former case, a practitioner would likely analyse the structure as-built in order to determine target areas for shear and flexural resistance enhancement. They would then design the strengthening intervention to either reduce the load carried by the MRF (e.g. by addition of lateral load carrying elements), or enhance the local and global seismic response of the MRF through the strengthening of structural elements (e.g. through addition of fibre-reinforced polymers or RC jacketing) and by moving the location of plastic hinges from the columns to the beams. In the case of RC MRF, both the determination of target deficiencies for strengthening intervention and the design of an effective strengthening strategy strongly depend on good understanding of the complex mechanical interactions in beam-column connections. This in turn requires specifically designed numerical and experimental studies that depict realistic structures. However, reviews of published experimental literature [1,2] show that most experimental studies on the seismic behaviour pre-1970's RC beam-column connections focus on the testing of sub-assemblies without slabs or transverse beams, which are unrepresentative of reality.

It is well-known (and recognised in modern seismic design codes) that transverse beams and slabs, significantly contribute to the response of beam-column connections under seismic loading. Despite this knowledge, it is here postulated that their effect on the response of beam-column connections, particularly in non-seismically designed RC MRFs, is underestimated in practice. It is suggested that this underestimation can derive from: (1) the use of empirical relationships for the calculation of slab width contribution to beam flexural capacity that are based on experiments on seismically detailed structures/beam-column sub-assemblages, and (2) for the case of 2D numerical analysis of the MRF frames, both the use of these effective flange widths and the neglect of transverse beam and slab effects on confinement of the joint. Importantly, in the context of structural assessment for retrofit design, this could result in the wrong target areas and mechanisms for strengthening being identified, potentially leading to ineffective strengthening interventions that do not rectify the weak-column strong-beam problem.

The few existing studies that include slabs and transverse beams in the set-ups all indicate the significant effect of slabs on the behaviour of the specimens, however these studies are either scaled-down specimens [3], represent corner joints [4] or are seismically designed containing significant joint reinforcement [5]. This paper presents the first part of a study being carried out to shed light on how (and by how much) slabs and transverse beams contribute to the seismic performance of deficient interior beam-column connections, and the consequences of ignoring or wrongly representing this contribution. It presents the results of experiments on three full-scale beam-column joints. One of these test specimens is designed to Eurocode 8 [6] and two are of typical pre-1970's design, with one specimen containing a slab and transverse beams, whilst the other represents the typical cross-shaped specimen tested in many other studies. The paper also presents preliminary nonlinear three-dimensional finite element (FE) models to complement the experiments, and extend them to look at the case of a pre-1970's specimen with transverse beams and no slab. As expected, the experimental and numerical results clearly demonstrate that the progression of damage and failure mechanisms in beam-column connections with and without slabs and transverse beams differ significantly. Generally, the results confirm inferences from the limited published experimental evidence [3,5] that the contribution of slabs and transverse beams can be underestimated by current guidelines.

2. Experimental Programme

A collaboration between University College London and University of Porto was set up in 2014 to conduct a large experimental programme of quasi-static cyclic tests on realistic full-scale interior beam-column connections. A first aim of these tests is to propose different strengthening approaches for realistic non-seismically designed interior joints (i.e. with slabs and transverse beams present), the details of which are published elsewhere [7–9].

The second aim is to provide an experimental base from which to start to investigate the complex interaction between all the elements framing into the joint. These experiments are discussed here.

2.1 Test specimens and materials

The three specimens presented in this study are designed to represent full-scale interior beam-column connections in a four-storey RC MRF. Each column is 1.5m long and has a square cross-section of 300mm by 300mm, representing half a storey. Each beam represents a half-span, is 2m long and has a rectangular cross-section of 450mm depth and 300mm width. The slab is 1.95m wide and 150mm deep.

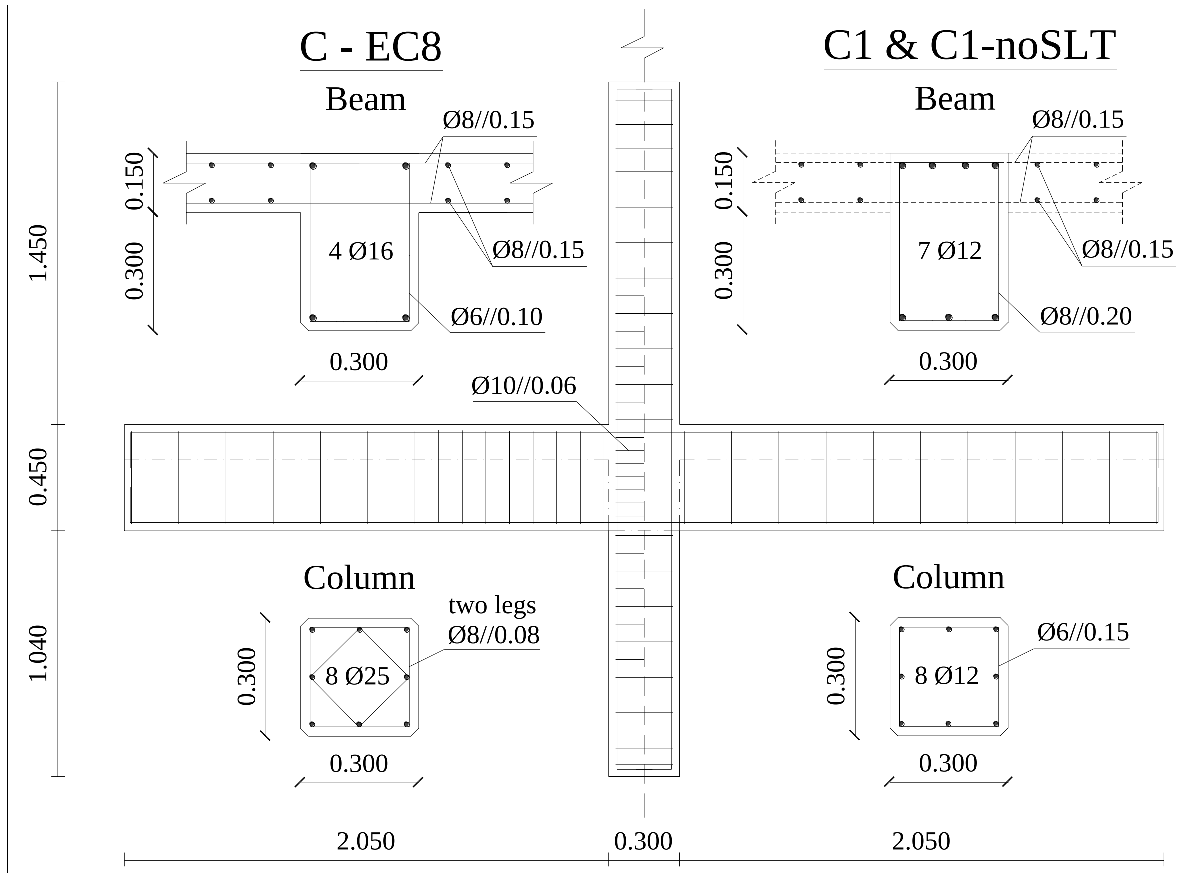

Fig. 1. Schematic of the geometry and detailing of the full-scale RC joint sub-assemblages tested. Note: the left side of the figure shows reinforcement details for specimen C EC8; the right side of the figure shows the reinforcement details for specimens C1 and C-noSLT (dashed line indicates slab for C1).

A schematic of the specimen geometry and reinforcement is presented in Fig. 1. The reinforcement detailing for one of the specimens, C EC8, is carried out according to Eurocode 8 provisions (Fig. 1 left). The steel reinforcement detailing of specimen C1 is instead designed according to the REBA (1967) code for reinforced concrete buildings in Portugal, chosen as representative of the codes of practice in place in southern Europe in the 1970's. This code shows several seismic detailing deficiencies common at the time, such as no transverse reinforcement in the joint, a lower flexural strength in columns as compared to beams, inadequate spacing of transverse reinforcement in columns for the provision of confinement (see Fig. 1 right). The third specimen, C-noSLT has identical detailing to C1 but does not comprise a slab or transverse beam (as denoted by the dashed lines in the beam section drawing), i.e. is of the cross shape commonly used in other studies. The average results of material tests on the three specimens are presented in Table 1.

Table 1. The mean concrete compressive strength of cylinder samples fcm, and the steel reinforcing bar yield strength fy and ultimate tensile strength fu for the three specimens tested.

Specimen	f_{cm} (MPa)	f_y/f_u - average (MPa)
C1	23.4	
C-noSLT	26.0	450/570
C-EC8	32.7	

2.2 Test Set-Up

The experimental set-up is the same for all the tested specimens and is presented in detail in [7], and so is not reproduced here. The loading set-up applies a lateral cyclic displacement protocol, with three cycles per increment, to the superior column of the joint sub-assemblage. A constant axial load of 425kN is applied to both columns, and an additional of 25kN is applied at the bottom to induce reaction forces in the beam supports, simulating moments from gravity loading. The principle beams are restrained near their ends with rollers, which impede their movement (locally) perpendicular to the longitudinal beam axis but allow them free movement parallel to this axis. The specimens are heavily monitored with strain gauges, LVDT's, rectilinear displacement transducers, draw-wire position transducers, inductive linear position sensors and cameras. The reader is again referred to [7] for further details.

3. Results and Discussion

Results from the three full-scale tests are presented and compared with results from numerical modelling of the specimens, as presented in [10]. A summary of the analysed parameters for the three specimens is given in Table 2. In this study, the yield drift, Δ_y, is based on the first strain gauge reading exceeding the steel yield strain (0.2%), and the ultimate drift, Δ_u, is related to a strength reduction of 20% (F_u) of the maximum force (F_{max}). The ratio of these drift values is expressed as the ultimate displacement ductility, $\mu_{\Delta u}$. E_{diss} is the total cumulative dissipated energy at the ultimate drift level. The initial stiffness, K_i, is defined as the slope between the origin and maximum drift in the first cycle. The global energy dissipation is calculated from the integral of the force-displacement curve. Finally, the ratio of the experimentally observed hogging moment capacity (M_{exp}) to the moment capacity predicted by Eurocode 8 (M_{EC8}) is provided for the beams in cases when the maximum moment is reached. Post-peak softening corresponds to the strength reduction observed for a specimen after reaching the maximum lateral load and is defined as the slope between the maximum force, F_{max}, and the ultimate force, F_u, at their respective levels of lateral displacement.

Table 2. Summary of main experimental results (difference to C1 in brackets).

Specimen	F_{max} (kN)	Failure location	Δ_y (%)	$\mu_{\Delta u}$	E_{diss} (kN.m)	K_i (kN/mm)	M_{exp}/M_{EC8}
C1	63.1	Superior Column	0.65	3.6	32.1	6.6	/
C-noSLT	45.16 (-28.4%)	Joint, Beams	0.59 (- 9.2%)	5.95 (+64.4%)	31.8 (-0.8%)	4.82 (- 27%)	0.99
C-EC8	123.9 (+96.4%)	Both Columns, Beams	0.85 (+30%)	6.1 (+69.1%)	172.3 (+437.1%)	7.18 (+ 8.8%)	1.35

3.1 Gravity-designed specimen with slab and transverse beam

For the specimen designed to pre-1970's guidelines, C1, numerical modelling predicts a brittle failure dominated by large rotation in the superior column, leading to a localised plastic hinge formation followed by concrete crushing and buckling of the superior column bars just above the column-slab interface [10]. The experimental observations shown in Fig. 2, confirm this. The type of mechanism observed can be described as a single-storey column failure, as no damage is observed in the rest of the specimen. As shown in Table 2, a relatively low lateral capacity of 63.1kN, combined with a low displacement ductility (3.6) are observed for the non-seismically designed specimen. The inadequate seismic performance of the specimen is also characterised by a very low energy dissipation (32.1 kN.m) and significant a post-peak softening slope.

Fig. 2. Final damage state in C1; left: superior column, right: beam underside

Analysis of the numerical model offers more in-depth understanding of the effect of the slab and transverse beam contribution. In Fig. 3, locations plastic strain in concrete, corresponding to cracking in the model, are shown. Damage is concentrated in the top column, at the joint interface, but high stains in the concrete are also observed at the interface between transverse beam and joint (inset), and interface of the slab and column, as the transverse beam cross-section is resisting joint deformation. Moreover, limited rotation of the primary beams is observed and their longitudinal reinforcement does not reach yield strain in the model and experiment alike. Yielding of reinforcing bars is limited to the columns (red colour in Fig. 3, right), and is observed at a drift of 0.65% in the experiment.

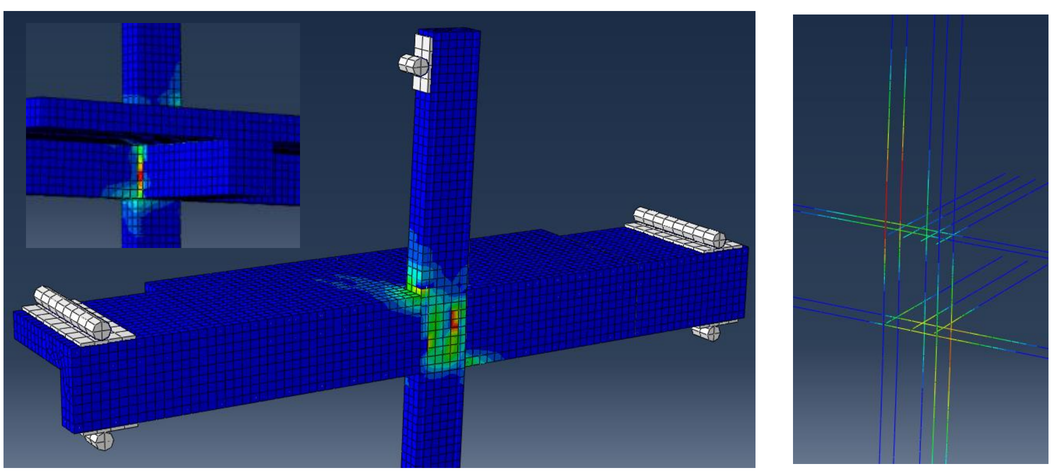

Fig. 3. Left - Damage location in control model with slab (Inset: view from the back); Right - yield of column bars – note: the displayed view is a cross-section through the centre-line of the interior joint specimen.

3.2 Gravity Designed specimen without slab and transverse beam

For the control specimen without slab and transverse beam, C-noSLT, a very different failure mechanism and damage pattern are observed. As shown in Fig. 4, and as predicted by numerical modelling, large damage occurs in the joint region, with cracks noticeable along the beam and at the beam-joint interface. Limited cracking in the columns and no yield of the column bars are observed, as the rotation of the column and hence the demand are reduced. This is in stark contrast to the column failure observed for C1.

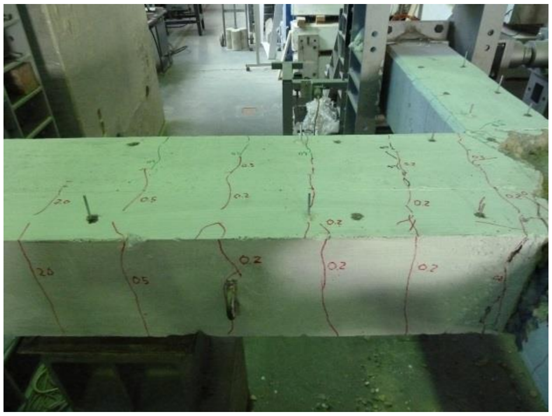

Fig. 4. Final damage state in C-noSLT; left: joint, right: beam

The effect of the slab is clearly highlighted by the evolutions of curvatures at the beam to joint interface shown in Fig. 5. In the case of C-noSLT the beam rotation is not limited by the slab and a symmetric curvature evolution is observed between positive and negative cycles (imposing sagging and hogging, respectively). Without the slab, the hogging moment capacity of the beam is similar to their sagging moment capacity and the beams and joint rotate. For C1, in turn, overall a much lower maximum curvature in the beams are recorded (- 67%), with the curvatures at the bottom of the beams three times larger than at the top.

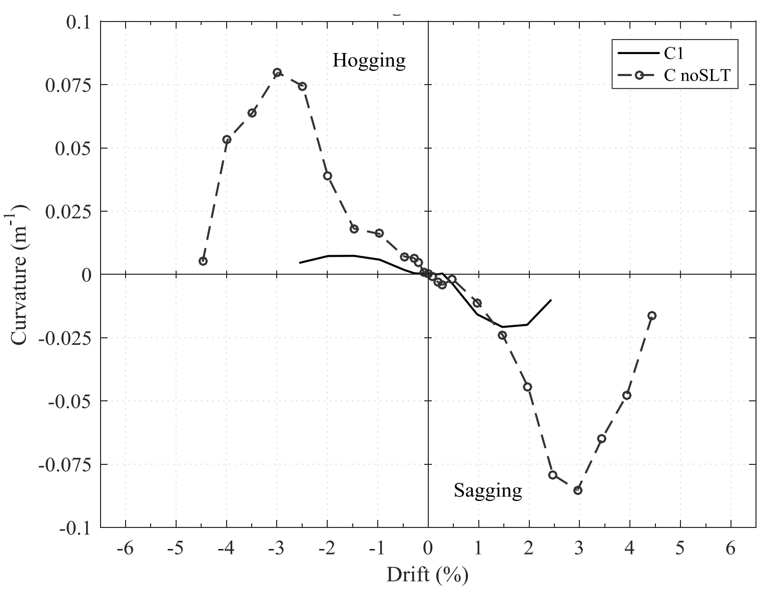

Fig. 5. Comparison of the experimental beam curvatures close to the joint interface for C1 and C-noSLT versus level of drift (%).

In terms of global force-drift envelope and energy dissipation (Fig. 6), specimen C-noSLT displays a much more ductile response, but with a much lower peak force (-28.4%) and a decreased energy dissipation (-0.8%). The increase in ductility is a direct consequence of the reduced yield drift and reduced initial stiffness. Due to the removal of the slab contribution, yielding is observed in the beams, and this leads to a lower yield drift of 0.59% (-9.2% compared to C1). As shown in Table 2, the initial stiffness of the specimen without slab is also much lower (-27%).

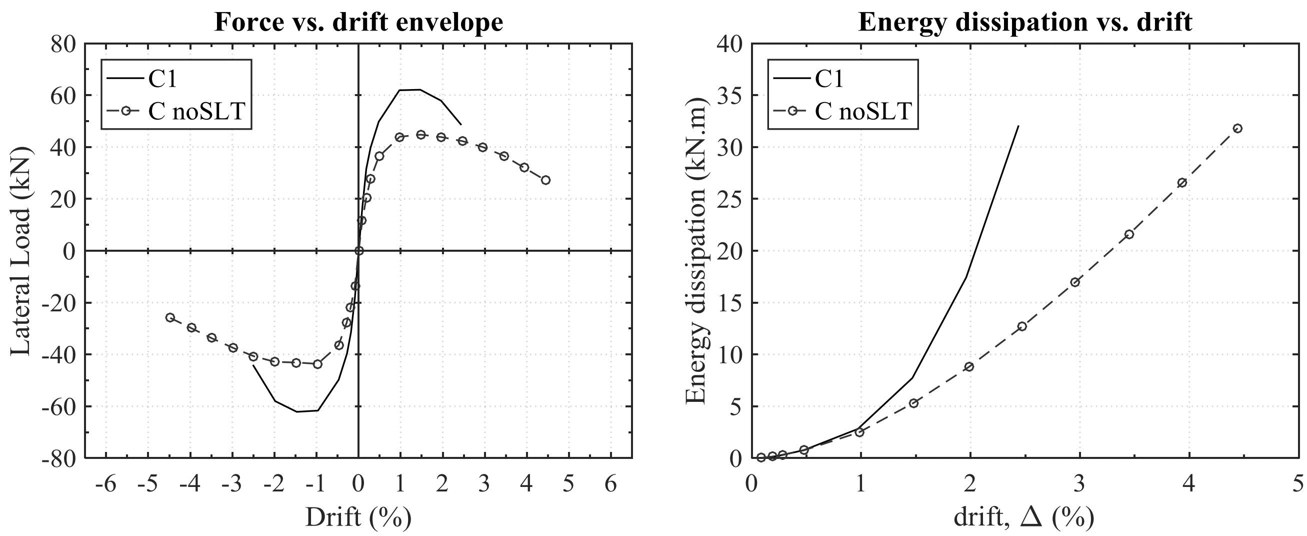

Fig. 6. Force-drift envelope and energy dissipation against drift for C1 and C-noSLT

While not tested in the laboratory, the isolated effect of the transverse beam is assessed by numerical modelling, as shown in Fig. 7. A specimen without slab but with transverse beams is modelled (Fig.7 right) and shows reduced damage as compared to the specimen without transverse beam (Fig.7 left). Furthermore, the transverse beam is seen to confine the joint and to also hinder the rotation of the joint, leading to larger damage in the adjacent beams. Finally, comparison of the behaviour of the numerical model of the specimen with transverse beams but no slab and C1 clearly shows the former having larger rotation and damage in the beams as opposed to the case with the slab present.

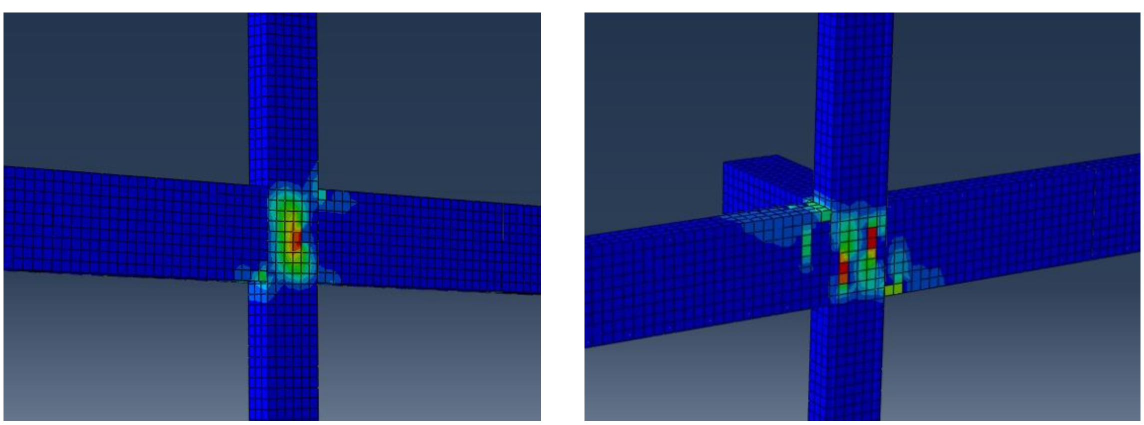

Fig. 7. Damage location in the specimen without transverse beam and slab (left) and the specimen without slab (right) [10]

3.3 Specimen designed to modern seismic guidelines (Eurocode 8)

The last specimen presented in this study is the control specimen designed to Eurocode 8, C-EC8. As expected, a much better cyclic performance is observed than in the gravity designed case. As shown in Table 2, a much higher strength of 123.9 kN (+96.4% compared to C1) and a larger ductility of 6.1 (+69.1%) are recorded for C-EC8. It can be seen in Fig. 8, that cracks are spread over a large area of both the beams and columns, leading to a more ductile failure.

The rotation of the columns is much more symmetric than for C1 and the high capacity of the column and joint ensure that yielding of reinforcement bars in the bottom of the beam (0.85% drift) precedes yielding of column bars (1.6%). A much improved dissipated energy (+437.1%) and reduced post-peak softening (-5.8%) are observed as a consequence of the improved seismic detailing.

However, the importance of considering the slab in the experiment is highlighted by the final damage state, shown in Fig. 8. Despite implementation of capacity design, concrete crushing in the columns is observed for C-EC8. This ultimately causes failure of the specimen. A higher over-strength of the beam bars may be one of the reasons for this failure of compliance with the anticipated hierarchy of strengths. However, the reason for this could also be attributed to a stronger slab contribution than anticipated by the Eurocode 8 design equations. Indeed the experimentally observed strength of the beams is 33% larger than anticipated by Eurocode 8 equations. It is worth noting that for the specimen without slab the prediction from EC8 is nearly perfect (-1%). In the numerical model, high strains in the slab longitudinal bars along the entire width of the slab are observed. A longer effective width of the slab, i.e. the t-section of the beam, (830 mm, i.e. 5.5 times the thickness of the slab) hence contributes to the moment capacity of the beam than what is anticipated in section 5.4.3.1.1 of Eurocode 8 (i.e. 4 times the thickness of the slab). This observation confirms results in [5] for which a capacity 39% higher than that calculated using seismic guidelines was also observed.

Fig. 8. Final damage state in C-EC8; left: superior column, right: beam underside

4. Conclusions

This paper presents preliminary results from a study that looks to assess the contribution of slab and transverse beams to the seismic behavior of full-scale interior beam-column joints typical of pre-1970’s design. Both the experimental observations and numerical simulations conducted show a large difference between the lateral capacities of interior joints with and without transverse beams and with and without slabs. This is expected, however, of concern is the observed difference in damage mechanisms. Without slab and transverse beam, the rotation of the beams is less constrained and the joint less confined, leading to observed damage in the joint and beams. The increased rotation of the beams, in turn, means that less demand is imposed on the columns, which consequently present no damage. This is the opposite of what is seen when transverse beams and slab are present. Where this is of concern is in the case when assessed structures are simplified as 2D frames in numerical analysis, where it is common to use effective flange widths to represent the slab contribution to the beam capacity and to neglect the confinement effects that transverse beams and slabs have on the joint. Importantly, in the context of structural assessment for retrofit design, this could result in the wrong target areas and mechanisms for strengthening being identified, potentially leading to ineffective strengthening interventions that do not rectify the weak-column strong-beam problem.

The results from the specimen with slab designed to Eurocode 8 confirmed previous observations in [5] that a significant part of the slab reinforcement contributes to the flexural capacity of the beams. The combined observations from experiments and numerical modelling suggest that the effective width of slab contribution may be underestimated in current guidelines. Underestimation of the slab contribution may have severe consequences to the seismic performance of RC MRF's as the hierarchy of strength with respect to capacity design principles may be reversed.

To confirm these observations however, the mechanisms involved need to be investigated further by means of a thorough numerical study and further experimental evidence. Moreover, the effect of bi-directional loading is not considered in this study and may play an important role in determining the real contribution of the slab to the overall beam and joint behavior [5].

5. Acknowledgements

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