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# Finite Element Modelling of a Reinforced Concrete Slab-Column Connection under Cyclic Lateral Load

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## Abstract

The behaviour of the reinforced concrete slab-column connection, adopted from a prototype flat slab system as subassembly model, is studied. To this end, a typical half-scaled reinforced concrete slab-column connection specimen was tested under cyclic lateral load up to 3.5% drift ratio. A simulation for a predicted model was then performed using finite element tool as a means to verify the results from experimental work and to provide a bigger picture regarding the initiation and progression of cracks. With regard to the behaviour of the test specimen, it is found that the performance of load carrying capacity is controlled by the formation of major cracks propagating alongside the slab surface. For comparative study, it is shown that the load-drift response of predicted model has a close resemblance to the test specimen. It is also shown that the longitudinal strain fields obtained from the numerical analysis are in good agreement with the crack pattern obtained from the experimental test.

**Keywords**: Reinforced concrete, slab-column connection, cyclic lateral load, cracking, numerical analysis

## INTRODUCTION

The attention of providing acceptable earthquake-resistant structure has been well-thought-out over the past few years. The majority of designing the earthquake-resistant structure does not only consider to improve the structural performance and occupant safety during the seismic events but also to construct the more economical system in the long run. It is worth mentioning that the earthquake action is a displacement oriented in nature; as of this sheer fact, the forces acting on the structure are controlled by the strength and stiffness. Therefore, providing such stable behaviour is important in order to prevent catastrophic failure. In reality, there have been many forms of building offered by engineers across the globe to build in the more cost-effective system, such as flat slab structure. This structure has become increasingly important as it offers more *economical system* and *distinctive characteristic* for not using any beam member (only slab and column). With regard to its response, nevertheless, the flat slab structure tends to undergo of what is typically known as punching shear behaviour due to low lateral stiffness.

Numerous collapses have been reported and investigated by several researchers over the past decades. The failure characteristic was indicated by the brittle punching shear which provoked the tendency to suffer the progressive damage. These several cases took place at L'Ambiance Plaza in Connecticut 1987 [1], Commonwealth Avenue in Massachusetts 1971, Skyline Plaza in Virgin Island 1973, and Harbour Cay Condominium Building in Florida in 1981 [2]. Other similar reported cases also took place in Mexico 1985 whereby 91 buildings collapsed severely due to the seismic events and also 40 buildings were heavily damaged [3].

In order to alleviate or even avoid the adverse side effect of punching shear failure, several studies concerning the behavioural improvement have been extensively carried out by researchers through the implementation of different material constituents such as the use of shear reinforcements on the slab [4, 5, 6, 7], the replacement of conventional concrete with fibrous materials [8, 9], the use of drop panel [10], the utilisation of post-tensioned [11], and so on – not just from these aspects, the understandings of general idea in relation to punching shear behaviour were also studied over many years ago [12, 13]. However, the majority of these studies were only concerned on mere experiments thereby causing the inefficiency if more specimens need to be constructed. In addition to that, the need for more testing is time consuming which also requires space and money, and for some reason, it might also be the case since the previous tests still have not triggered the significant changes in any design codes. At this point, the idea of performing the response of structural member, with a bigger picture, can be indispensably done through finite element simulation. Although it seems convenient, modelling the specimen in the level of simulation is quite challenging particularly when considering the nonlinear behaviour. Therefore, it is necessary to ensure whether or not the numerical simulation is appropriate. To deal with this, the verification of numerical results against experimental data must be undertaken prior to simulating the further impending models. It also should be noted, the input parameters from verified model to proposed model must, at least, analogous or only have tiny differences with experimental results in order to maintain a close resemblance with regards to the behaviour of actual specimen.

The emphasis of this research is to propose a reliable technique for evaluating the behavioural response of reinforced concrete

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slab-column connection through numerical analysis. The specimen is prompted to cyclic lateral load as it represents the earthquake force in reality. The specimen is designed based on the typical shape of flat slab system with the supplementary attachment of drop panel member as local thickening with the tenacity of reducing the negative moment occurred at the joint as well as to improve the lateral stiffness. Prior to simulating the predicted model using finite element tool, the behaviour of the test specimen is initially discussed. For the purpose of providing a properly predicted model, the numerical simulation is undertaken utilising the same material and structural constituent as the test specimen. The selection of appropriate constitutive model is primarily important as it affects the way in which the predicted model behaves during the phase of loading. Not only does it influence the computational load but it also can lead to the contradictory results if it is not properly considered. Through this study, the overall behaviour of predicted model is justified by the results from finite element analysis.

## TEST SPECIMEN

The test specimen was a half-scale model of an interior slab-column connection using drop panel with the detail of geometry shown in **Figure 1(a) and (b)**. The test specimen was designed without any shear reinforcement on the slab, yet it was also considered to have low flexural reinforcement ratio for inducing the failure mechanism in under-reinforced section. This is essential as it provides better behaviour by letting reinforcement to yield before concrete crushing. The flexural reinforcement of slab was also designed continuously by passing through the column to maintain the integrity of slab and column during the course of high loading. Detail of reinforcement layouts on the top and bottom of the slab can be seen in **Figure 1(c)**. During the test procedure, the cyclic lateral load was applied at the column tip. The supplementary of constant low gravity load, calculated from the superimposed load and 30% of live load, was also applied to the top surface of the slab. The linear summation of these loads resulted in 0.1 gravity shear ratio.

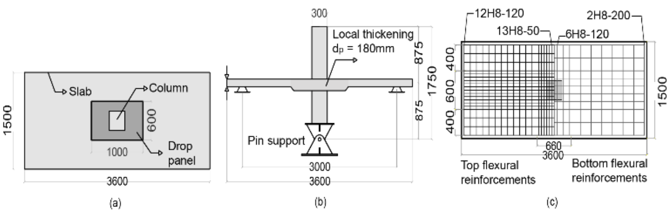


**Figure 1:** Detail of test specimen: (a) plan; (b) side view; (c) reinforcement layouts (unit in millimetre).

*Test setup and instrumentation*– The specimen was tested using the rig in accordance with the testing configuration shown in **Figure 2**.

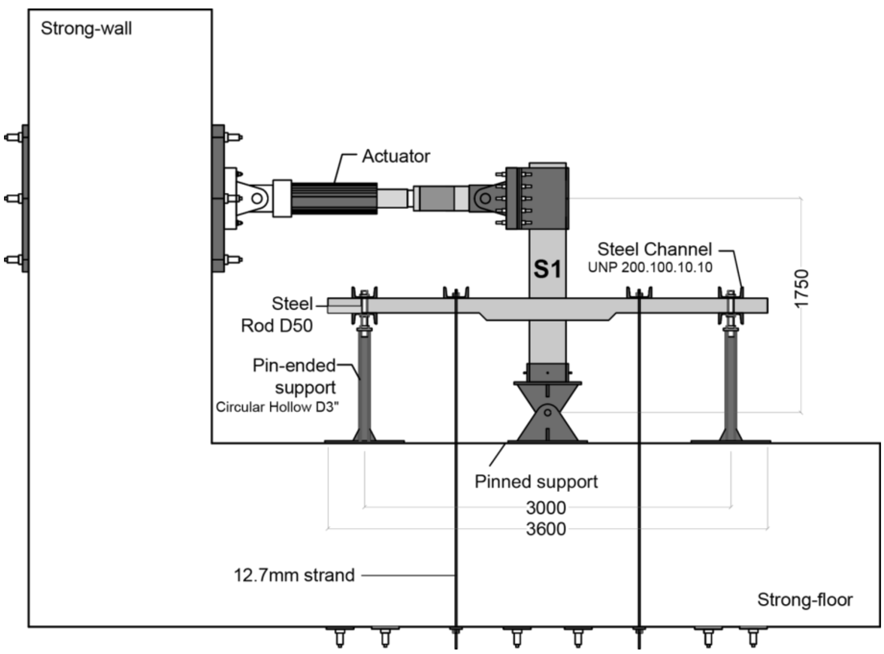


**Figure 2:** Testing configuration of slab-column connection (unit in millimetre).

Upon moving the specimens into the location of the test, all supports were attached and fastened to the strong floor. After completing this, the specimen was lifted up using an overhead crane and was then moved along to the testing rig. The underneath surface of the column was attached to pin support. Both of slab edges were also restrained using pin-ended vertical roads to prevent any vertical displacement, yet allowing a free lateral movement and rotation. In the middle of testing preparation, it was also important to consider whether or not the test specimen in the upright and straight position for the purpose of obtaining the proper data acquisition. Once the specimen remained stable, the constant gravity load was then given over the surface of the slab following by the application of cyclic lateral load at the column tip in the form of displacement control. Detail of lateral displacement routine used in this experiment is shown in **Figure 3**.

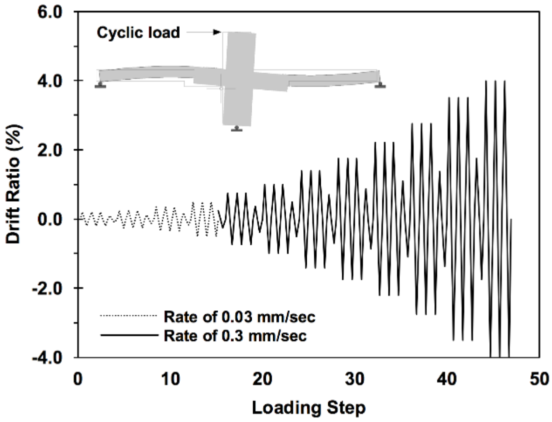


**Figure 3:** Displacement routine for test specimen.

*Material properties*– The concrete used to make this specimen was supplied by the local concrete company with a specified mean compressive strength of 47.7 MPa. During the casting process, there standard concrete cubes 150-mm were also cast and cured along the test specimen where they were placed in a shed outdoor environment to represent the common site practice. The same as concrete, the reinforcing bars were also supplied by the local company with the different grade for each size. A total of 29-Nos of slab longitudinal reinforcing bars, 44-Nos of slab transversal reinforcing bars, 12-Nos of column longitudinal bars, and 35-Nos of stirrups, were used in this study. Representation of each size of these reinforcements was tested under uniaxial tensile load, resulting in the tensile properties summarised in **Table 1**.

**Table 1:** Tensile properties of reinforcing bars

| **Rebar type** | **Bar size** (mm) | **Area** ($mm^2$) | **Elastic region** | | **Inelastic region** | |
|---|---|---|---|---|---|---|
| | | | $f_y$(MPa) | $\varepsilon_y$(%) | $f_u$(MPa) | $\varepsilon_u$(%) |
| Threaded | 12.78 | 128.01 | 394.89 | 0.197 | 499.90 | 13.61 |
| Plain | 7.96 | 46.42 | 324.87 | 0.217 | 480.50 | 11.65 |
| Plain | 6.89 | 37.27 | 327.07 | 0.192 | 480.39 | 10.62 |

*Load-drift relationship* –The relationship of lateral load carrying capacity and drift level of the test specimen, also characterised as a hysteretic curve, is shown in **Figure 4**.

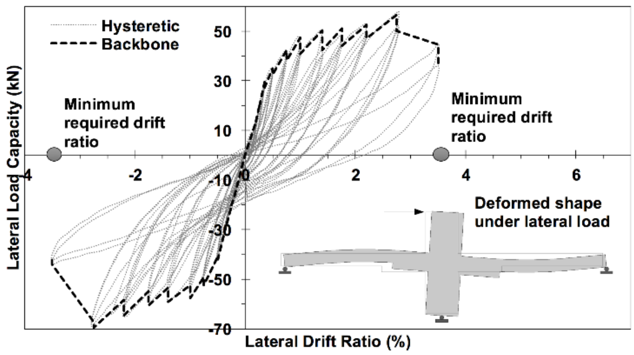


**Figure 4:** Load-drift relationship of test specimen

Within this Figure, the overlaid envelope (backbone) curve is also included to assess the single response of lateral load and driftin each drift level. In general term, the specimen exhibits sufficient ductile plateau as indicated by inelastic deformation prior to failure. This is associated with the yielding of slab flexural reinforcements thereby prompting the specimen to engender the inelastic deformation. During testing, it was observed that the initial response was linear. As the loading increased, the reduction of lateral stiffness was evident followed by the initial progression of cracks to take place at the critical section around the connection. It was observed that the first cracks appeared at ~4.38mm drift level. As the load continued to increase beyond the post-cracking zone, a significant transitional line was apparent implying the response of the specimen to deviate from the linearity. At this point, the response was in the post-yielding zone whereby the progression of significant cracks over the top and bottom surface of the slab was also evident. When the response had achieved the peak load, the discernible crack formation, increased in length and width, was apparent followed by the large opening of crack bands, particularly in the critical zone and interface between drop panel and slab. As the load further increased in excess of the peak point, spalling of concrete underneath the surface of drop panel was apparent causing the specimen to suffer the strength degradation. Throughout the loading process, it was shown that failure mode was in flexure. In the matter of this case, it can be said that the likelihood of specimen to achieve the flexural capacity prior to punching failure might happen.

## FINITE ELEMENT SIMULATION

For the purpose of verifying the result, a nonlinear finite element simulation was undertaken to model the similar specimen as examined in the experimental test. The numerical study was based on the use of the well-known commercial finite element software ABAQUS [14] as it offers an appropriate applicability of carrying out 3D static, quasi-static, or dynamic nonlinear simulations. In order to describe the concrete element, the *solid hexahedral*, incorporating the eight-node linear brick with reduced integration (C3D8R), was used herein. It was purposely preferred as it provides a solution of equivalent accuracy in terms of resulting in proper stress and strain values without taking a too heavy computational load. Not just that, this element also tends to be not stiff in bending; hence it is very suitable for concrete. Another advantage of using this element is that it can reduce the mesh sensitivity during the iteration course. For reinforcing bars, two-node truss element (T3D2) with3D *wire* was employed, where the effects of reinforcement bars were homogenised over the concrete. To account the perfect bond between concrete and reinforcement, embedded constraint technique was adopted. When the reinforcing bars are embedded, the transitional degrees of freedom of will automatically be eliminated since they have merged with concrete nodes.

*Concrete damage plasticity* –To account the nonlinearity in material properties of concrete, the concrete damage plasticity (CDP) was adopted in this present work. CDP is a continuum, plasticity-based, damage model which assumes two failure mechanisms consisting of the compressive crushing and tensile cracking (see **Figure 5**).

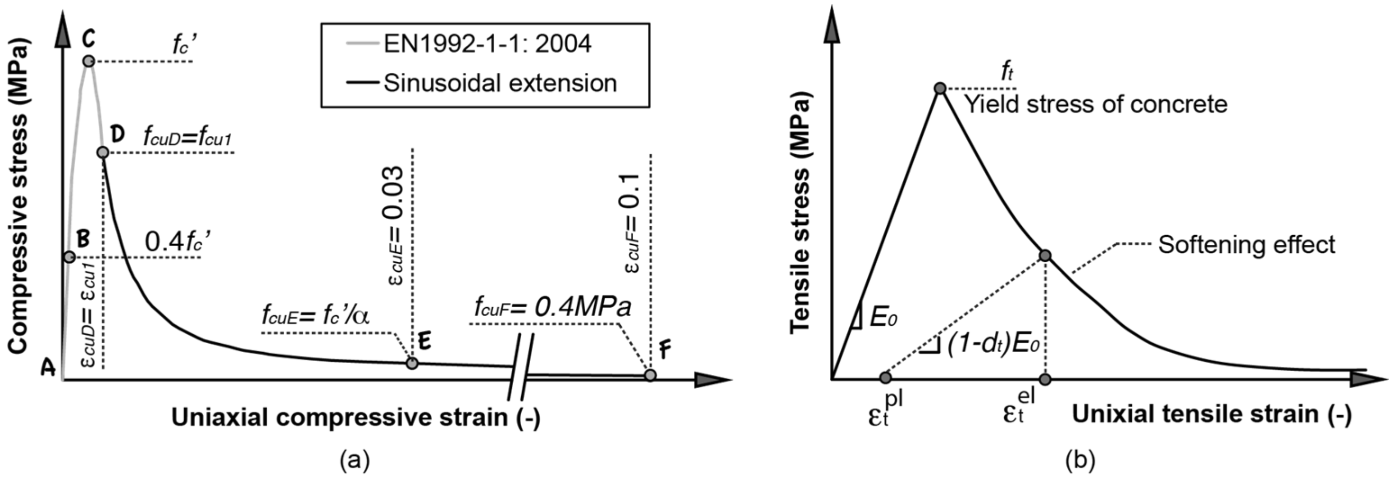


**Figure 5:** Parameters of concrete damage plasticity: (a) compression behaviour; (b) tension behaviour.

In general, the main concepts of continuum mechanics are effectuated to simulate the complex nonlinear behaviour of quasi-brittle material (e.g. concrete) as a means to evaluate the structural behaviour when putting this material all together to construct the structural members. In addition to that, the concepts of continuum mechanics are projected to provide a better prediction in relation to structural response for any type and level of loading – not just from this perspective, the concepts are also anticipated for assessing the nonlinear behavioural response of structural members especially for high loads (*e.g.* earthquake and impact). With regard to the adequacy of adopting the concepts of continuum mechanics, CDP has also been assessed as one of the best models to represent the complex behaviour of concrete by implementing isotropic damaged elasticity in combination with isotropic compressive and tensile plasticity for accounting the inelastic behaviour. Since the major challenge of modelling the structural member in the finite element tool lies in the understanding of how to input the proper material properties, especially when concerning the nonlinear response; therefore, it is important to consider the parameters used in the constitutive models to obtain the most accurate result. Given this situation, the following paragraph describes the way to obtain such proper parameters in relation to material properties of CDP.

For modelling CDP in the finite element programme, four criteria that must be taken into account are the plasticity parameters, compressive behaviour, tensile behaviour, and damage state. To obtain the specific values for each criterion, substantively, the concrete or any quasi-brittle material must be tested under uniaxial, biaxial, and triaxial tests. However, it is not necessary these days; as of the fact, there have been extensive studies done by researchers to propose different approaches for providing the reliable CDP parameters. One particular approach that has shown a good agreement in the practical use is suggested by Pavlovic *et al.* [15]. The equations are simply straight forward thereby enabling the ease in the calculation process. For more detailed information regarding the equations used in this approach, the readers are referred to Reference 15. For plasticity parameters, the dilation angle $\Psi$ is considered 36 degrees, the shape factor $K_c$ is 0.667, the stress ratio $\sigma_{b0}/\sigma_{c0}$ is 1.16, the eccentricity $\varepsilon$ = 0.1, and the viscosity parameter $\mu$ is 0.01. Concerning the damage state in the CDP input, the damage value for compression and tension is in a range of 0 to 0.99.

*Normal plasticity* – For the purpose of inputting the material properties of reinforcing steels regarding its nonlinear response, normal plasticity was used herein with the adoption of isotropic hardening model. Using this feature, it means that the yield surface remains in the same shape but expands with increasing or decreasing stress. In most cases with regard to the tensile behaviour of reinforcing bars, however, the trending curve in the hardening zone tends to increase gradually. To provide a simple, yet accurate model, the uniaxial tensile stress-strain relation for reinforcing bar embedded in concrete is assumed to be in the bilinear curve whereby the initial response is linear elastic until yielding then perfectly plastic. The idealised stress-strain responses of reinforcing bars used in this study is seen in **Figure 6**.

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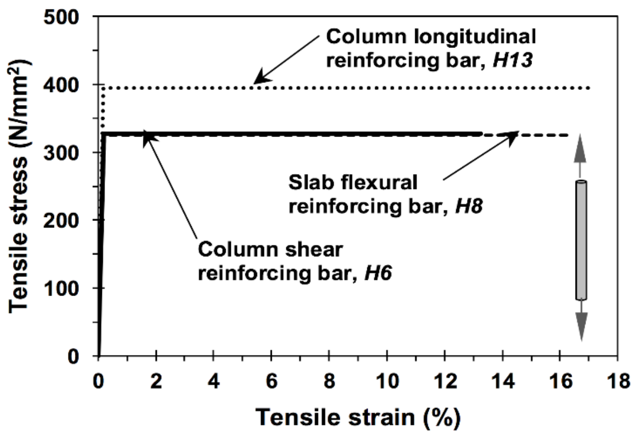


**Figure 6:** Assumed average stress-strain relationship of reinforcements embedded in concrete

*Boundary condition and loading etiquette*–Described in briefly, the boundary condition is utilised in the numerical model as a means to define the degrees of freedom at nodal supports. For pin support in the column, nodal points were restrained in the translational (U) direction of U1, U2, U3 while the rotational directions are left unchecked. In the matter of slab supports, nodal points were restrained underneath the slab edges by preventing any movement in all translational and rotational direction except for U1. In the simulation procedure, constant gravity load was applied to the specimen in the form of pressure load prior to subjecting the model to the lateral load. Furthermore, following the same procedure as experimental work, the cyclic lateral load was given using displacement control but with only one cycle for each increment of drift level to hasten the computation time.

## RESULTS AND DISCUSSION

*Load-drift relationship* –As mentioned earlier, the emphasis of undertaking the finite element simulation is to provide in-depth insights regarding the behavioural response between test specimen and predicted model. Given that, a series of slab-column connection was simulated in the finite element software under cyclic lateral load. The comparison between observed and predicted load-drift relationship, shown in **Figure 7**, is discussed herein. It is shown that the comparison of backbone curves is in a good agreement to one and another, meaning that the simulation was carried out in the proper manner. With regard to predicted model, the load-drift response can be divided into three main stages:

(i) an initial response where the specimen behaves in linear elastic;

(ii) a change in gradient curve following by the alteration of stiffness at the load level of ~40 kN thereby causing the response to deviate from its linearity,

(iii) descending line as a means of post-peak response where the specimen suffers the loss of strength.

Throughout the loading course, it can be sighted that the predicted model exhibits a small measure of ductility as indicated by the loss of strength prior to 3.5% drift level.

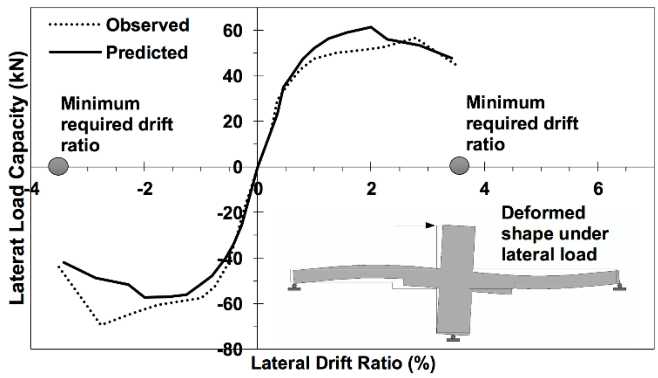


**Figure 7:** Comparison of observed and predicted load-drift responses (backbone curves).

For the purpose of discussion, a comparison of observed and predicted load-drift responses is presented in **Figure 7.** It is shown that in the initial response, a similar linear response of observed and predicted specimen is evident; however, as the load is in the post-yielding zone, the significant discrepancy is obvious. This indication may be driven by a phenomenon to what is called as *hourglass effect* acting on the linear brick elements. As the concrete adopted the reduced integration elements, the integration points lie along the centred vertical place in an eight-node brick element; hence, as having *hourglass effect*, this element could wrap into a trapezoidal shape from a rectangular shape without the integration point experiencing any stress. In addition to that, *hourglass effect* is also associated with spurious deformation which leads to inaccurate results in the nonlinear region if the model has a *significant* deformation. Considering all the above, needless to say, ABAQUS, unfortunately, is unable to generate the pinching effect in the hysteretic curve [16,17]; therefore, it is intentionally neglected to be displayed along the backbone curve of predicted model.

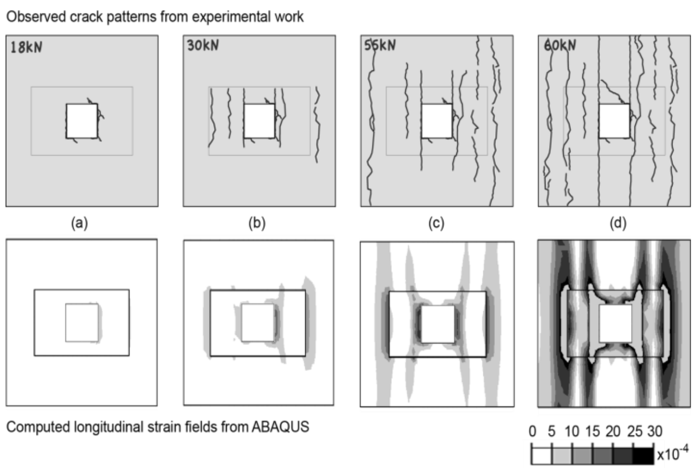


**Figure 8:** Comparison of computed strain profiles and crack pattern under different stages of loading.

*Crack pattern* – To provide a bigger picture into the full behaviour of reinforced concrete slab-column connection, the initial formation and propagation of strain profiles of predicted model, otherwise portrayed as a crack pattern, are presented in **Figure 8**, emphasizing the onset of cracking (~18kN), the post-cracking response prior to yielding (~30kN), the post-yielding response before peak load (~55kN), and the peak point response (~60kN). As shown in the Figure, it is evident that in the initial response, there is only a subtle development of strain strips during the early phase of loading whereby this situation is associated with the onset of cracking. As the loading progresses around the post-cracking zone, the longitudinal strain profiles begin to propagate with the increase in length and width. As the load further increases beyond the yielding level, the significant crack propagation is apparent whereby the extension of crack tips have reached the slab edges. When the specimen achieves the peak load level, a discernible increase in strain band widths is obvious owing to the fact that the longitudinal strain profiles have reached the nominal capacity of 0.003. At the last state of crack pattern, it is visibly shown that the predicted strain profiles resemble the actual crack pattern of the observed test specimen shown in **Figure 8**.

## CONCLUSION

A simple, yet applicable, finite element modelling has been undertaken to study the behavioural response of reinforced concrete slab-column connection under cyclic lateral load. For the considered specimen, the comparative study between experimental and numerical results are in good agreement thereby implying that constitutive models in the numerical model are capable of providing accurate predictions on the overall behavioural response in the reinforced concrete slab-column connection. A comparison of load-drift relationship represents the similar trend in the linear and post-cracking response up to yield level. The slight change in structural response throughout the nonlinear phase is essentially linked to the *hourglass effect* thereby making the predicted model deviates from the resemblance. In another aspect, it is also proven that the longitudinal strain profiles of predicted models are in the same trend of observed test specimen, owing to the fact the finite element modelling is done through the appropriate technique.

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