ANALYTICAL STUDY ON SEISMIC PERFORMANCE OF PARTIALLY PRESTRESSED CONCRETE BEAM-COLUMN JOINTS

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ABSTRACT

In this study, six partially prestressed concrete exterior Beam-Column Joints with variable prestressing force and four partially prestressed concrete interior beam-column joints with variable column axial load ratio have been analytically analyzed and assessed to evaluate their hysteretic performance under reversed cyclic loading. A Two-Dimensional finite element software program, VecTor2, is used to validate the non-linear response of beam-column joint experiments executed in Chiba and Kyoto university, Japan. The analytical result adequately simulated the interior joints in all cycles of loading while in the exterior joint a reasonable underestimation of results was obtained at the last cycle. In the partially prestressed concrete exterior beam-column joints, variation of prestressing force had little effect on the ultimate storey shear capacity. Stiffness and ductility increased significantly with prestressing force before wide shear crack formation and high prestress loss at the joint. Strength degradation after peak response was severe with increasing prestress level. This phenomenon undermined the inelastic energy dissipation capacity of the highly prestressed joints at the later cycles. The increment of column axial load in the partially prestressed concrete interior beam-column joint resulted in wider pinching while the converse created severe pinching. Premature crushing of concrete at the joint due to high compressive stress from the column axial load and prestressing force was not observed in any of the specimens.

Keywords: Partially Prestressed Concrete, Beam-Column joint, Prestressing force, Column axial load ratio, VecTor2, hysteretic response.

INTRODUCTION

For many years, the use of prestressed concrete members has been accepted to be advantageous for structures under flexure since it counteracts externally applied gravity loads. However, after a while, their significance in primary seismic-resistant members such as frames and shear walls has created a substantial argument. In Nishiyama's research [1], a partially prestressed concrete exterior beam-column joint, prestressed with $0.5f_{yp}$ ($f_{yp}$is the yield strength of prestressing steel), was subjected to a reverse cyclic loading and showed improved hysteretic performance than ordinary reinforced concrete joints. However, these joints were designed to fail in flexure with plastic hinge occurring at the beam-column interface. Thus, the true shear behavior of the joint was not fully understood. Kashiwazaki and Noguchi [2] studied the effect of prestressing force level on the ultimate storey shear capacity of partially prestressed concrete interior beam-column joints and concluded that no significant effect was obtained.

According to Paulay and Priestley [3], many beam-column joint failures have been observed in the 1980 EI Asnam [4] earthquake. Shear and anchorage failures, particularly at exterior joints, have also been identified after the 1985 Mexico [5], the 1986 San Salvador [6], and the 1989 Loma Prieta [7] earthquakes. Beam-column joints are very critical regions in reinforced concrete frames designed for inelastic response to seismic excitation. This is because they are located in an area, where shear and bond stresses are considerably high. These forces are resisted by diagonal compression strut mechanism and truss mechanism. The diagonal compression strut resistance mechanism is fully active before the stress transfer mechanism at the joint is demolished due to degradation in bond strength. The performance of a diagonal compression strut mechanism can be improved by confining the joint. Usually, joint confinement is provided by adding more transverse reinforcements. In addition to that, as discussed in Park and Paulay [8], compressive stress from column axial load widens the diagonal strut region in the joint as a result of an enlarged compression block across the column region. Due to the formation of a wider diagonal compression strut, horizontal bond forces along the longitudinal beam bars can now be disposed of more easily.

In partially prestressed concrete beam-column joints, additional joint compressive stress is provided from the compression force due to post-tensioning. By considering this advantage, in this study, the effect of introducing compressive stress in the partially prestressed concrete exterior beam-column joint with variable prestressing force is studied and their hysteretic behavior is evaluated. Premature crushing of concrete in partially prestressed concrete interior beam-column joint, that might occur due to high compressive stress, obtained from column axial load and prestressing force is also covered in this study.

1. Finite element modeling and Validation

Description of the examined partially prestressed concrete joints

Two experimental programs, partially prestressed concrete interior and exterior beam-column joint, executed by Kashiwazaki and Noguchi [2], and Nishiyama and Wei [9] in Chiba and Kyoto University, Japan, were used to validate the analytical result. Both joints are designed to fail in shear at the joint to evaluate the joint's ultimate capacity. The interior specimens were subjected to a constant column axial load of 320 kN while no column axial load was applied to the exterior specimens. Figure 1 and Figure 2 show the sectional detail and reinforcement layout of partially prestressed concrete interior and exterior beam-column joints. Table 1 and Table 2 shows the material properties of the partially prestressed concrete interior and exterior beam-column joints.

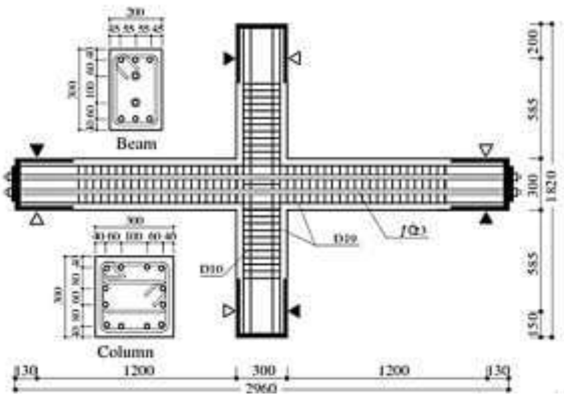


Figure 1 Sectional detail, reinforcement layout, and test setup of the partially prestressed concrete interior beam-column joint

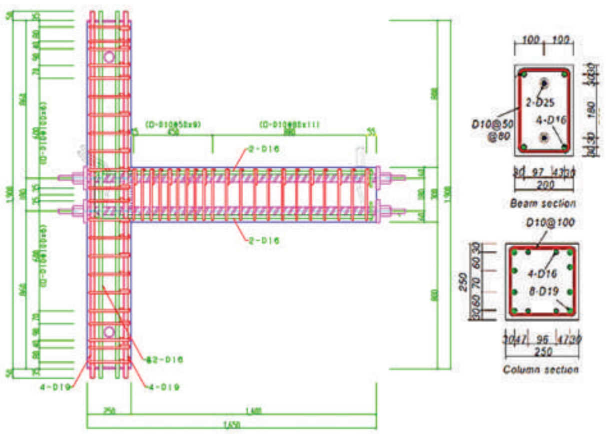


Figure 2 Sectional detail, and reinforcement layout for partially prestressed concrete exterior beam-column joint

Table 1 Material properties of partially prestressed concrete interior beam-column joint specimens

a) Concrete				
Specimen	Compressive strength, f_c (MPa)	Elastic modulus, E_c (GPa)	Split strength, f_t (MPa)	Prestress (MPa)
PC-0	34.4	28	3.01	0
PC-1	34.4	28	3.01	$0.25f_{yp}$
PC-2	35.3	28.1	3.01	$0.5f_{yp}$

b) Reinforcement and Prestressing steel					
Type of reinforcement	Diameter (mm)	Yield strength, f_y or f_{yp} (MPa)	Yield strain, ε_y (mm/m)	Elastic modulus of steel, E_s (GPa)	Ultimate tensile strength, f_u (MPa)
Longitudinal	19	517	2.85	181	692
Transverse	10	897	4.33	207	1070
Prestressing steel	23	1100	5.5	200	1250
N.B: f_y is yield strength of reinforcement and f_{yp} is yield strength of prestressing steel					

Table 2 Material properties of partially prestressed concrete exterior beam-column joint specimen

a) Concrete					
Specimen	Compressive Strength, f_c (MPa)	Stain at f_c, %	Elastic Modulus of concrete, E_c (GPa)	Split tensile Strength, f_t (MPa)	Prestress (MPa)
KPC2-1	34.6	0.22	28.2	2.51	$0.59f_{yp}$

b) Reinforcement and Prestressing steel					
Type of reinforcement	Diameter (mm)	Yield stress, f_y / f_{yp} (MPa)	Yield strain, ε_y (mm)	Elastic modulus of steel, E_s (GPa)	Ultimate tensile strength, f_u (MPa)
Transverse	10	307	1.744	176	436.9
Longitudinal	16	374.7	2.025	185	533.4
Longitudinal	19	386.8	2.113	183	569.9
Prestressing steel	25	1026	5.104	201	1146
N.B: f_y is yield strength of reinforcement and f_{yp} is yield strength of					

Material modeling

The developed finite element model sufficiently captured the material nonlinearity of concrete, reinforcement, prestressing steel, and the bond characteristic between the steel and the surrounding concrete by assigning the appropriate material models for each material property. A bond link element was assigned between the reinforcement/ prestressing steel and the surrounding concrete, at the critical regions of the partially prestressed concrete exterior beam-column joint.

Their bond-slip behavior was defined by assigning the appropriate confinement pressure index factor according to the equation provided under F. J. Vecchio et al [10]. The reinforcements in the interior joints were assumed to be perfectly bonded since the confinement pressure provided by the transverse reinforcements exceed the high confinement pressure value (7.5 MP_a) accepted by Committee Euro-International Du Beton, CEB-FIP Model Code 90 [11]. Concrete regions were modeled by using a rectangular concrete element and transverse reinforcements were defined as smeared elements inside the confined regions of the concrete by defining the appropriate transverse reinforcement ratio. Reinforcement and prestressing steel regions were modeled as truss bar elements.

Mesh, constraints and loading

In the partially prestressed concrete interior beam-column jointa rectangular mesh type was used with an element size of 40mm by 40mm in all regions except the load-bearing region at the beam ends and top and bottom of the column, for which is 23x40mm and 25x40mm were used. Four load cases were defined. Load case 1 was a monotonic loading which was assigned for the constant column axial load and the own weight of the specimen. Load case 2, 3 and 4 was a reverse cyclic loading that represents the lateral storey displacement history, $\pm 7, \pm 14, \pm 21, \pm 28, \pm 45$ and $\pm$72mm, which was applied progressively at the beam ends. Moment release was provided at the end of the beams and at the top and bottom of the column to simulate the inflection points in the actual structure. A push and pull load was assigned at the tip of the beam to simulate load reversal due to alternative drift. Figure 3 shows loading and boundary conditions for this specimen.

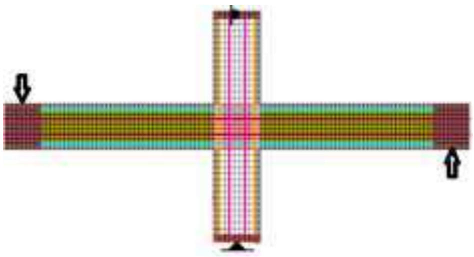


Figure 3 Loading and boundary conditions

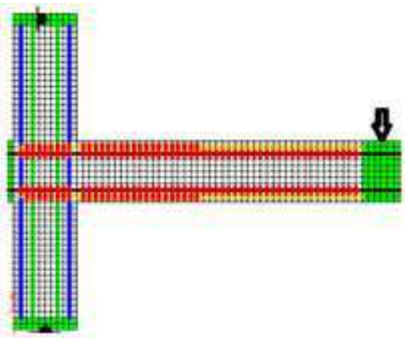
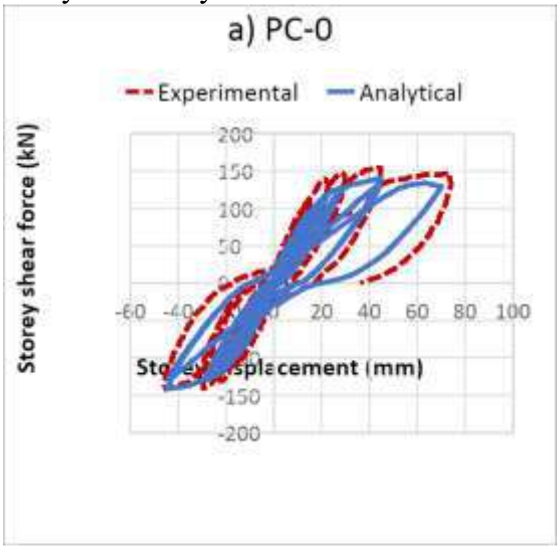


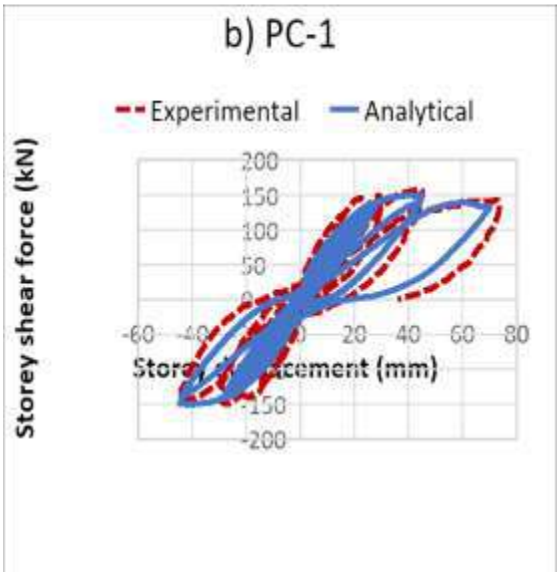
Figure 4 Bond link element, loading and boundary conditions

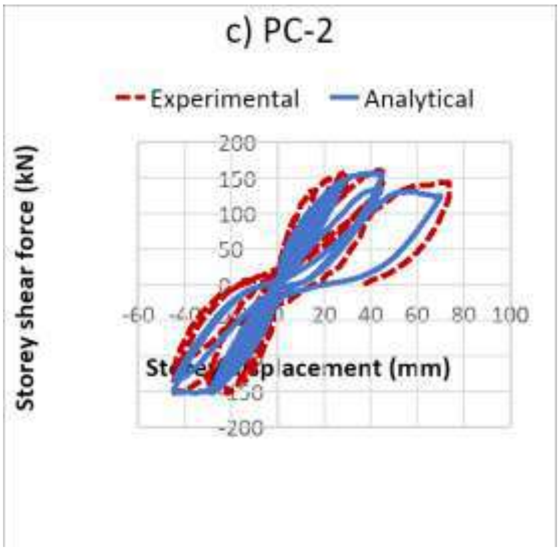
In the exterior joint a rectangular mesh was used with an element size of 25x30mm at the beam region and 30x30mm for the rest of the regions. Three load cases were defined. Load case 1 was a monotonic loading which was assigned for the self-weight of the specimen. Load case 2 and 3 was a reverse cyclic loading that represents the lateral storey displacement history, $\pm 12, \pm 24, \pm 36, \pm 48, \pm 60$ and $\pm$90mm, which was applied progressively at the beam ends. Figure 4 shows bond link element, loading, and boundary conditions for these specimens.

Analytical and Experimental result comparison

In general, the finite element model sufficiently simulated the experimental result in both partially prestressed concrete interior and exterior specimens. The analytical hysteretic curves for the interior specimens (PC-0, PC-1, and PC-2) and exterior specimens (KPC2-1) in comparison with the experimental result are presented in Figure 5. The error that occurred in specimen KPC2-1 at the last cycles of the loading was significant relative to the others. This is due to the software's inadequate performance when the specimen's material nonlinearity becomes more pronounced as the number of repeated loading cycles increases after peak capacity. Such errors are believed to be improved by executing a three-dimensional analysis. Since two-dimensional models do not capture the effect of out-of-plane crack propagation, they tend to exaggerate surface crack width which might in turn underestimates the capacity of the member. The analytical failure crack pattern observed at each specimen in comparison with the experimental is also presented in Table 3.







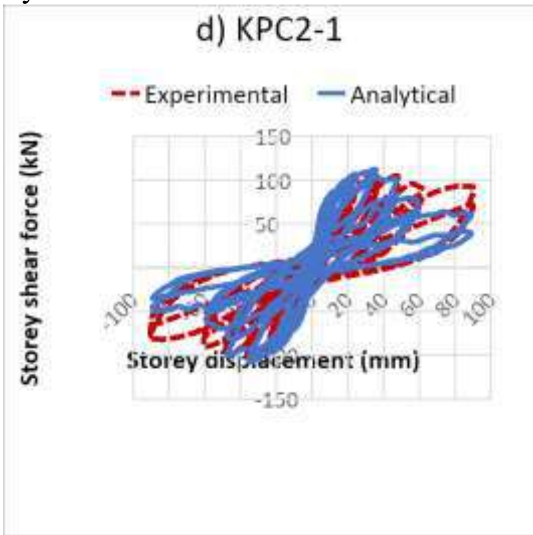


Figure 5 Analytical & Experimental hysteretic response comparison for interior and exterior specimen

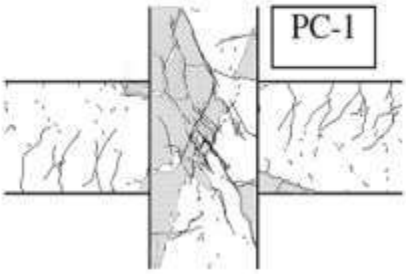
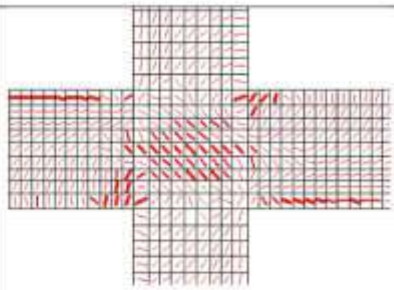
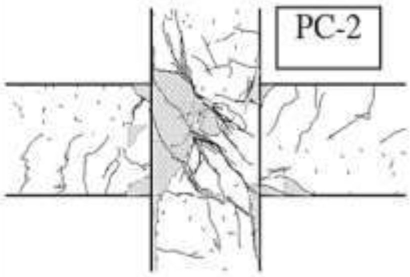
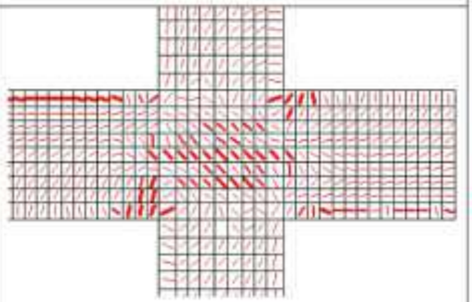
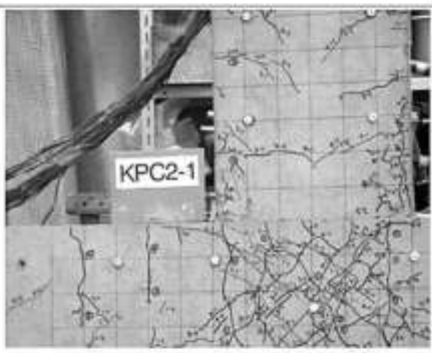
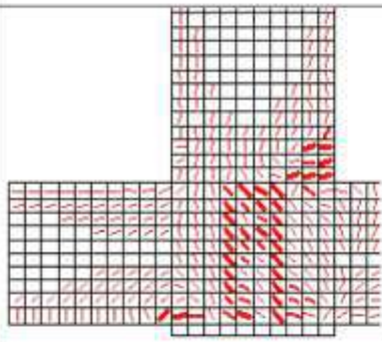
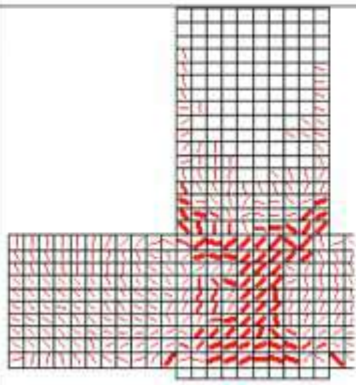
1. Parametric analysis, result, and discussion

Parametric analysis

In this study two parameters were studied, prestresseing force level and column axial load. The effect of prestressing force level was first assessed by analyzing the previously validated partially prestressed concrete exterior beam-column joint by varying the applied prestress in the prestressing steel, holding other parameters to be constant. The specimen details were defined according to their prestressing force level and presented in Table 4.

On the other hand, the effect of column axial load on a highly prestressed joint was evaluated by analyzing four partially prestressed concrete interior joints. The specimens were subjected to an axial load ratio of 0, 0.1, 0.4, and 0.65, with a constant prestress of $0.8f_{py}$ as shown in Table 5.

Table 3 Failure crack pattern comparision for the experimental and analytical model

Specimen	Experimental	Analytical
PC-1 (at a storey displacement of 74mm)		
PC-2 (at a storey displacement of 74mm)		
KPC2-1 (at a storey displacement of 36mm)		
KPC2-1 (at a storey displacement of 90mm)		

Analytical result

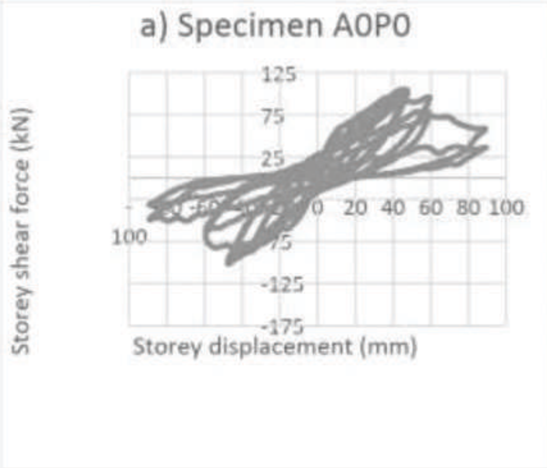
The seismic performance of the joints was evaluated based on the storey shear force - storey displacement hysteretic response. The storey displacement is the displacement equal to the lateral displacement history applied at the tip of the beam and the storey shear force is the lateral reaction at the tip of the beam under the applied lateral displacement. Figure 6 shows the storey shear force vs storey displacement hysteresis response of partially prestressed concrete exterior joints. Figure 7 shows the hysteretic response for partially prestressed concrete interior joints.

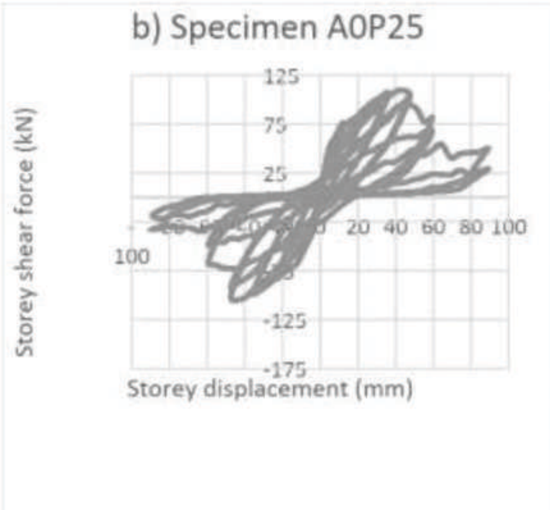
Table 4 Summary of prestress loading on partially prestressed concrete exterior joint

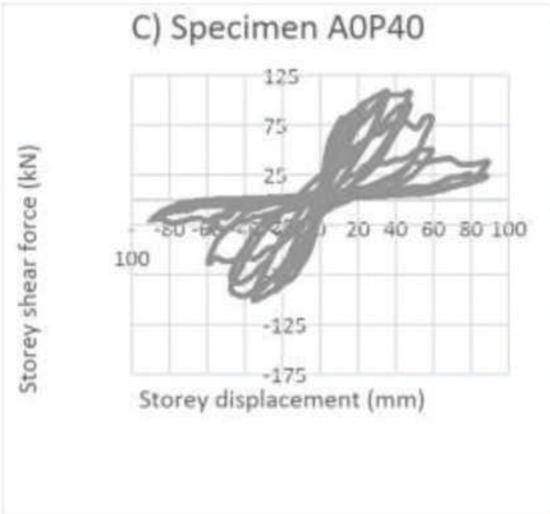
Specimen	A0P0	A0P25	A0P40	A0P59	A0P70	A0P80
Prestress ratio (%)	0	25	40	59	70	80
Prestress σ_{pg}(MPa) (%fpy)	0	256.5	410.4	615.6	718.2	820.8
N.B: f_{py} is Prestressing steel (PS) yield strength						

Table 5 Summary of column axial load on partially prestressed concrete interior joint

Specimen	IA0P80	IA10P80	IA40P80	IA65P80
Column axial load, N_{ED} (kN)	0	320	1238	2012
Axial load ratio, $N_{ED}/(A_c*f_{cd})$	0	0.1	0.4	0.65
N.B: A_c is the gross sectional area of the column and f_{cd} is the design compressive strength of concrete				







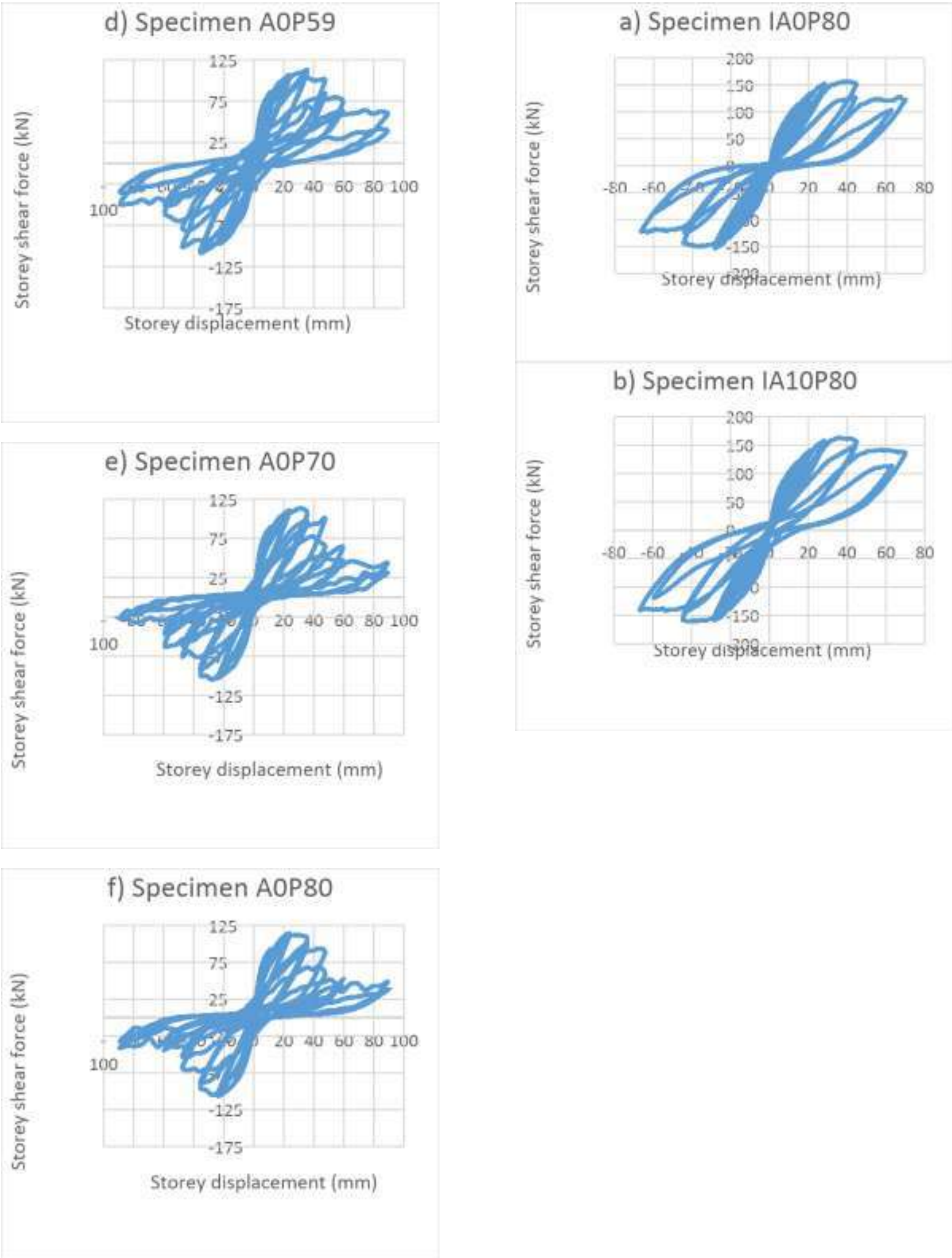


Figure 6 Hysteresis response of partially prestressed concrete Exterior beam-column joint

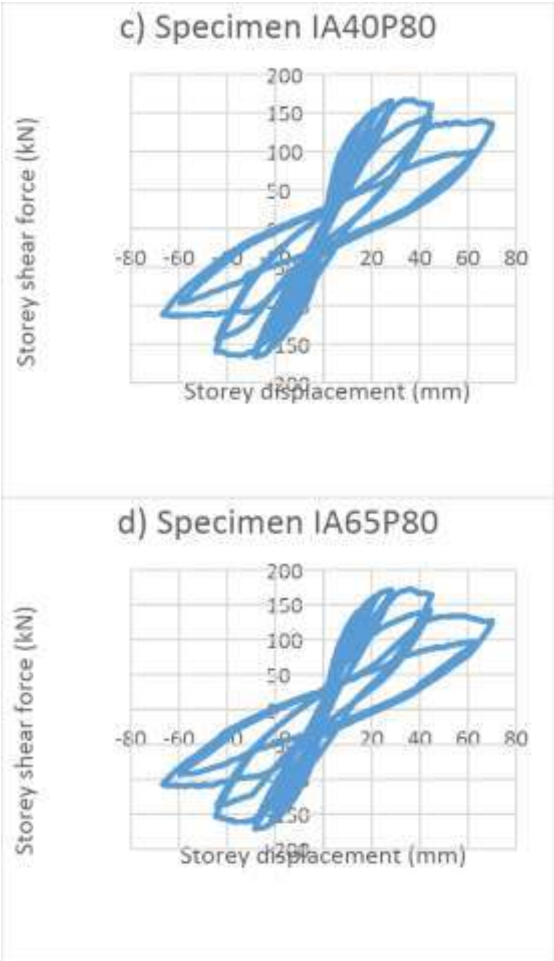


Figure 7 Hysteretic response of partially prestressed concrete Interior beam-column joint

DISCUSSIONS

I. Effect of prestressing force on the partially prestressed concrete Exterior beam-column joint

The seismic performance was evaluated based on storey shear capacity, stiffness, shear strength degradation after peak response, ductility, absorption, and energy dissipation capacity Ultimate storey shear capacity.

(a) Ultimate storey shear capacity

As can be observed from Figure 8, the prestressing force resulted in an insignificant effect on the ultimate storey shear capacity, under both positive and negative loading protocol. This is because under higher cyclic displacement prestressed concrete sections resemble the behavior of reinforced concrete sections. Therefore, they will have more or less similar capacity after experiencing some cycles.

(b) Ductility, Stiffness, Strength degradation, and Energy dissipation capacity

High ductility is essential in seismic design to delay the local failure of members by allowing plastic redistribution of actions from one critical section to another and to allow absorption and dissipation capacity of the input energy.

Figure 9shows the effect of prestressing force on the displacement ductility factor. It can be seen that the increase in prestressing force in the beam-column joint resulted in a substantial increment in the ductility of the specimen. This is mainly due to the significant variation in the yield displacement. Due to the initially applied compressive stress by the prestressing steels, highly prestressed joints were able to counteract the positive deformation from the external loads at the initial stages of the loading. This led joints with higher prestress level to achieve their yield and peak/ultimate capacity at smaller yield and peak/ultimate deformations (Δ_y *and* Δ_{peak}) as shown in Table 6.

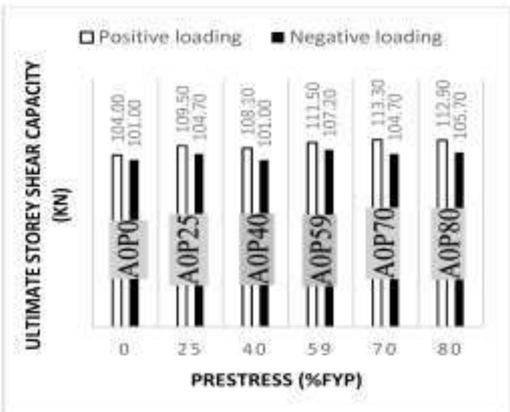


Figure 8 Effect of prestressing force on the ultimate storey shear capacity of partially

prestressed concrete Exterior beam-column joint

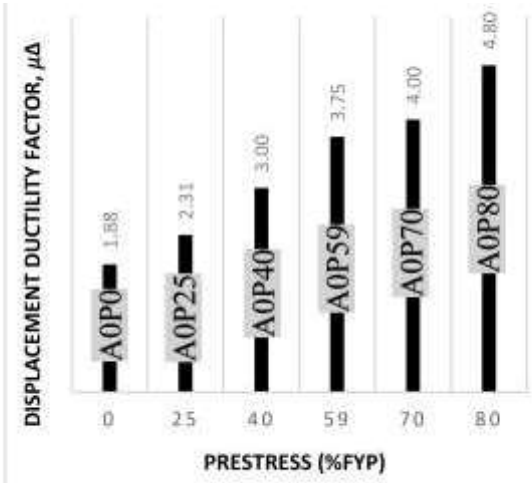


Figure 9 Effect of prestressing force on displacement ductility factor

Table 6 Yield and Ultimate displacements of partially prestressed concrete exterior joint

Specimen	A0P0	A0P25	A0P40	A0P59	A0P70	A0P80
Δ_y , (mm)	32	26	20	16	12	10
$\Delta_{@75\%Peak}$, (mm)	60	60	60	60	48	48
Δ_{peak} , (mm)	48	48	36	36	36	24

Due to similar reasons, joints with higher prestress levels achieved greater stiffness as shown in Figure 10. Thus, they were able to attain their capacity without undergoing excessive deflection. On the contrary, the effect was completely reversed at the later stages of loading (after the 4th cycle). This was due to substantial loss of strength after peak response which resulted from prestress loss and was more pronounced for highly prestressed joints.

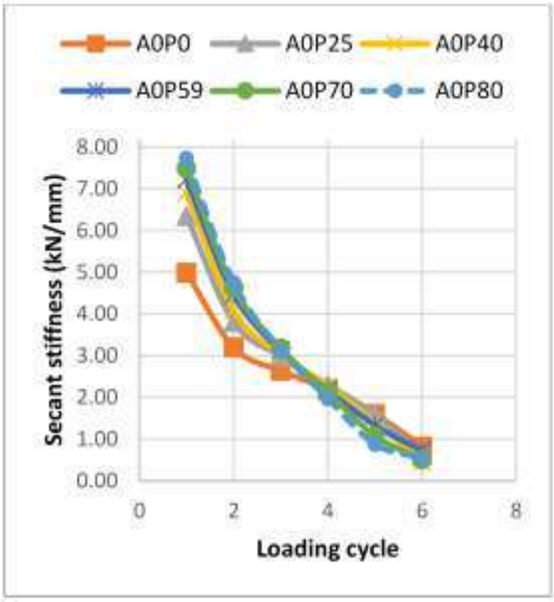


Figure 10 Secant stiffness of exterior joints, computed for each loading cycle

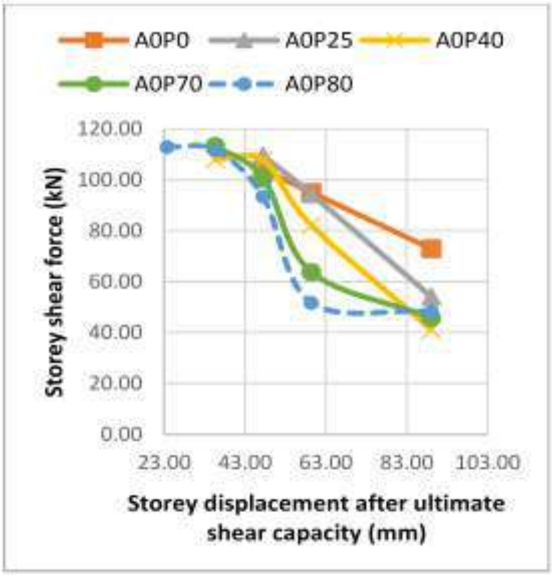


Figure 11 Strength degradation after peak response

The strength degradation after peak response and loss of prestress is shown in Figure 11 and Figure 12. These phenomena undermined the energy dissipation capacity of joints with higher prestress levels at the later cycles of the loading as shown in Figure 13.

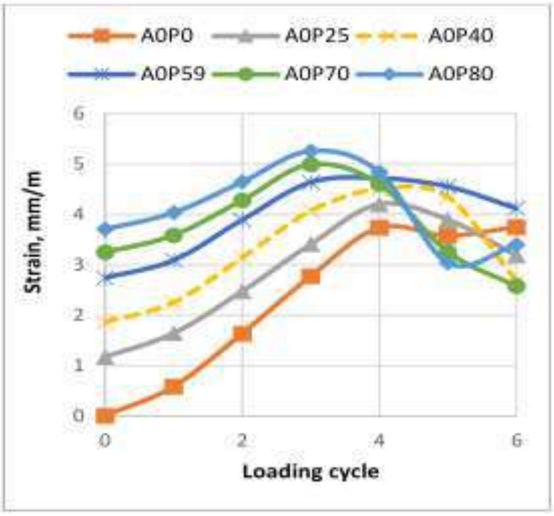


Figure 12 Strain distribution in the prestressing steel at the joint for each loading cycle

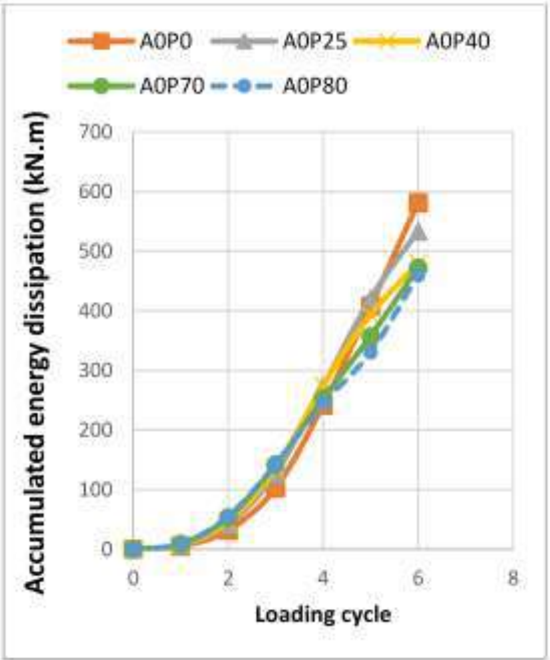


Figure 13 Effect of prestressing force on accumulated energy dissipation capacity

Effect of column axial load on the partially prestressed concrete Interior joints

The seismic performance of these specimens was evaluated based on premature crushing of joint concrete due to high compressive stress from the column axial load and prestressing steel. The column axial load ratio effect was also evaluated on the Pinching behavior of the joints.

(a) Premature crushing of concrete joint

To study the premature crushing of concrete joint two parameters were taken into consideration. The first one is the ultimate storey shear capacity and the second one is an observation of internal vertical concrete stresses at the joint that is obtained at the last cycles of the loading in specimen IA65P80. Figure 14 clearly shows the effect of column axial load on the ultimate storey shear capacity of the specimens. Based on the analysis result, column axial load increment slightly enhanced the shear capacity of the joints. This is because the incoming compressive stress from the column axial load restrains the joint region against shear failure which considerably improves bond performance by preventing slippage of reinforcement bars.

In addition, an average concrete compressive stress observed in the beam-column joint at the final loading stage of specimen IA65P80 was 15MPa which is 56.3% of the compressive strength of the concrete (34.4MPa).

Thus, since no drop of ultimate strength was encountered with column axial load increment and the incoming stress is still much less than the compressive strength of concrete at the joint, crushing of concrete is not expected to occur. Therefore, premature crushing of concrete due to high compressive stress from the column axial load and prestressing steel did not take place.

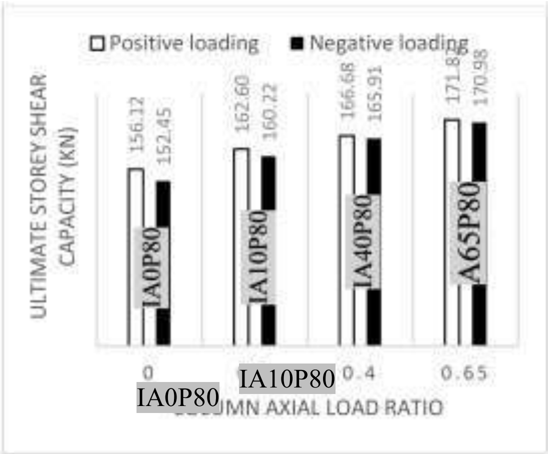


Figure 14 Effect of column axial load on the ultimate storey shear capacity

(b) Pinching
In reinforced concrete structures subjected to reverse cyclic loading, a large reduction in loading stiffness after unloading might occur. In the reloading branch of a hysteresis curve, after repeated cycles, cracks that occur previously in the tension side might still be open in addition to the one that is going to be created at the new face under tension. As a consequence, the concrete at the section will be fully cracked and the concrete will be ineffective in resisting the shear. This phenomenon causes narrowing of the hysteresis loops. Such effect is known as pinching. The column axial load tends to offset the pinching behavior of the interior joints. As shown in Figure 7, specimen IA0P80, with no column axial load, experienced severe pinching. But as the column axial load increases, a moderate pinching was observed in the specimens. In specimen IA40P80 and IA65P80, the high compressive stress from the column axial load delayed the formation of shear crack at the joints as can be seen from Figure 15. Therefore, since majority of these joint regions were not cracked immediate recovery of stiffness was possible.

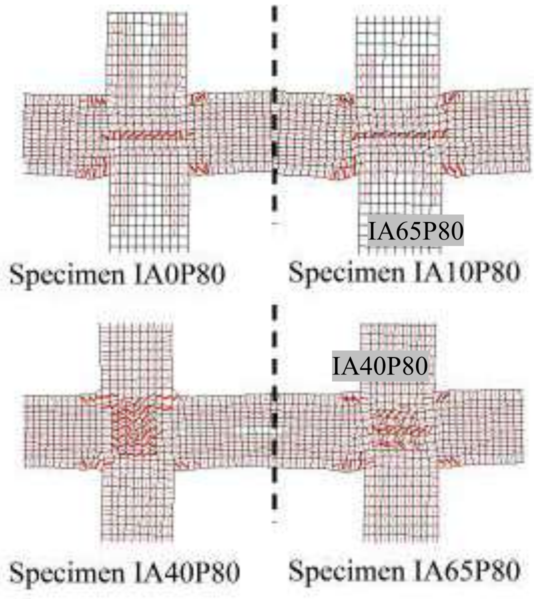


Figure 15 Failure crack pattern of the interior joint at the final stage of loading

CONCLUSIONS

In this study analytical investigation on the effect of prestressing level and column axial load on the seismic performance of partially prestressed concrete exterior and interior beam-column joint was studied. According to the analytical result, variation of prestressing force did not encounter a significant effect on the ultimate shear capacity. Although, a significant increment in the ductility and stiffness of the joint was observed, this effect was completely reversed after the 4th loading cycle due to significant degradation of strength following the loss of prestress.

This phenomenon undermined the energy dissipation capacity of joints with higher prestress levels at the later cycles. On the other hand, in the interior joint specimens, column axial load ratio variation in a highly prestressed joint did not encounter premature crushing of concrete at the joint at any level of the loading. Furthermore, a less pronounced pinching was observed with column axial load increment.

REFERENCES

[1] Nishiyama, M. "Seismic design of prestressed concrete buildings," Bulletin of New Zealand National Society for Earthquake Engineering, vol. 23, no. 4, pp. 288–304, 1990.

[2] Kashiwazaki T. and Noguchi H., "Structural performances of prestressed concrete interior beam-column joints," Proc. 12th World Conference Earthquake Engineering (12WC EE), Auckland, New Zealand, January 30-February 4, 2000, pp. 1–8, 2000.

[3] Paulay T. and Priestley M. J. N., "Seismic Design of Reinforced Concrete and Masonry Buildings," Second edition JOHN WILEY AND SONS, INC., 1992.

[4] Berter V.V., and Shadh H., EI Asnam, Algeria Earthquake, October 10, 1980, Earthquake Engineering Research Institute, Oakland, Calif., January 1983, 190p.

[5] Mitchell D., "Structural damage due to the 1985 mexico earthquake," Proceedings of the 5th Canadian Conference on Earthquake Engineering, Ottawa, 1987, A. A. Balkema, Rotterdam, pp. 87-111.

[6] Caltrans Seismic Design References, California Department for Transportation, Sacramento, Calif., June 1990.

[7] Loma Prieta Earthquake October 17, 19 Preliminary Reconnaissance Report, Earthquake Engineering Research Institute, Oakland, Calif., 50 p.

[8] Park R. and Paulay T., "Reinforced concrete structures," First edition JHON WILEY & SONS, INC., 1975.

9] Nishiyama M. and Wei Y., "Effect of post-tensioning steel anchorage location on seismic performance of exterior beam-to-column joints for precast, prestressed concrete members," PCI Journal, vol. 52, pp. 18–30, 2007.

10] Wong,P. S. Vecchio F. J., and Trommels, H. "VecTor2 and formworks user's manual," Second Edition 2013.

[11] CBE-F IP Model Code 90, Institute of Civil Engineers (ICE) Publication., pp. 82-116