Studies on nonlinear behavior of retrofitted reinforced concrete beam column joints after experiencing severe damage from earthquake load simulation

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Abstract. For an earthquake resistant structure, reinforced concrete building must have certain performance level under certain level of earthquakes such as when it is subjected to a strong level earthquake, it may experience severe damages, but without partial or full collapse, thus some reparations could be done to recover the functions of those damaged structures. However, repairing methods were usually done to slightly-damaged structures, while for severely-damaged structures, more studies are still needed to optimize the effectivity of the repair. Therefore, the objective of this study is to evaluate the performance of a structure that is retrofitted using high strength concrete after experiencing severe damage from an earthquake. Reinforced concrete beam column joints – that are used as specimens for this study – were initially subjected to cyclic loading up to 5% drift. The specimens' beams are then repaired by replacing the damaged concrete with the new, stronger concrete without replacing the existing reinforcement bars. The retrofitted specimens are then subjected to the same cyclic loading and their nonlinear behaviors are compared to the behavior of their initial condition. The experimental results show that there are mostly reductions in lateral strengths, although there is an increase of strength in one specimen, while there are also reductions in energy dissipated.

1 Introduction

Earthquake is one of the many things' engineers should consider when it comes to design a structure, especially in an area that is prone to earthquakes. When designing an earthquake resistant structure, reinforced concrete buildings must achieve certain performance level under certain level of earthquakes. Structures must be able to be occupied immediately when they are subjected to frequent earthquakes. When they are subjected to design earthquakes, they may experience moderate damages, but they must be functional again after reparation. However, when they are subjected to strong earthquakes (maximum considered earthquakes), they may experience severe damages but without partial, neither full, collapse. Damaged structures can be repaired to function as they are intended or demolished so it shall not be used anymore. While not treated, their capacity will be lower than their initial capacity according to the damage taken during an earthquake [1].

There is a lot of research that studies on earthquake damaged buildings' treatments. One study that is done by J.H. Kim (2015) states that there are 233 damaged reinforced concrete buildings that are observed after the Christchurch Earthquake. From the observation, a total of 138 buildings are demolished, 65 are retrofitted, and 20 are still unknown, therefore stating that most damaged buildings tend to be demolished instead of repaired with various reasons [2].

Researches about retrofitting structures have also been done many times, mostly by reinforced concrete jacketing. This retrofit procedure is done by adding another layer of reinforced concrete, confining the repaired elements, thus making them stiffer and stronger [3,4]. However, increasing one element's capacity, especially the beams, can alter the mode of failure of a structure, causing this method to be ineffective against a damaged beam structure. Therefore, different retrofit procedure needs to be done on each specific case to be effective. One method that can be studied is by replacing the damaged concrete with newer concretes, so that the capacity of the beam does not significantly increased, hence does not alter the structure's mode of failure. This research's objective is to measure the effects of retrofitting severe damaged reinforced concrete beam column joints by replacing the damaged concrete with

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high strength concrete. The effects that are measured are the changes in structure's behavior, such as its lateral strength, deformability, and energy dissipation capacity.

2 Methods

2. 1 Preparation

Two non-fiber beam column joints from previous experiments by Kurniawan et. al [5], as shown in Fig.1, are investigated after severely-damaged by earthquake simulation. It is known that both specimens are using high strength reinforcing bars with a yield strength around 500 MPa. From the investigation, it is found out that both specimens' beams experienced spalling near the joint, while both columns only experiencing small hairline cracks, causing only for the beams to be repaired. For ensuring that there are no damaged concretes in the specimens' beams, concrete that is located around 500 mm from the column's face is removed. The surfaces of the substrates are roughened up to ensure good bond between the new concrete with the substrate. Both specimens are then straightened out so that they can be retested again as beam column joints. After ensuring the beam is perpendicular to the column, high strength concrete grout of 55 MPa is then put onto the specimens, replacing the removed ones. Before putting in the concrete, water is poured onto the substrate to ensure the surface is wet enough for the reparation. Cylinder tests of the concrete grout are also done to ensure the strength of the concrete. Specimens are then cured using wet sack to keep the concrete moist and kept for at least 28 days before being tested.

2.2 Cyclic testing

Both specimens are then set to an actuator to be tested as shown in Fig. 22 where the bottom column is restrained to a hinge support. Both ends of the beams are also restrained, but to a roller support, to ensure the specimens behave as a joint. Tests are done by applying a displacement controlled cyclic load, as shown in Fig. following the module defined in ACI 374.2-13, to the top of the column. Initial specimens are named as SD16-0 and SD19-0 for specimens with 16 mm diameter longitudinal bars and 19 mm diameter longitudinal bars respectively, while repaired specimens are named as SD16-R and SD19-R.

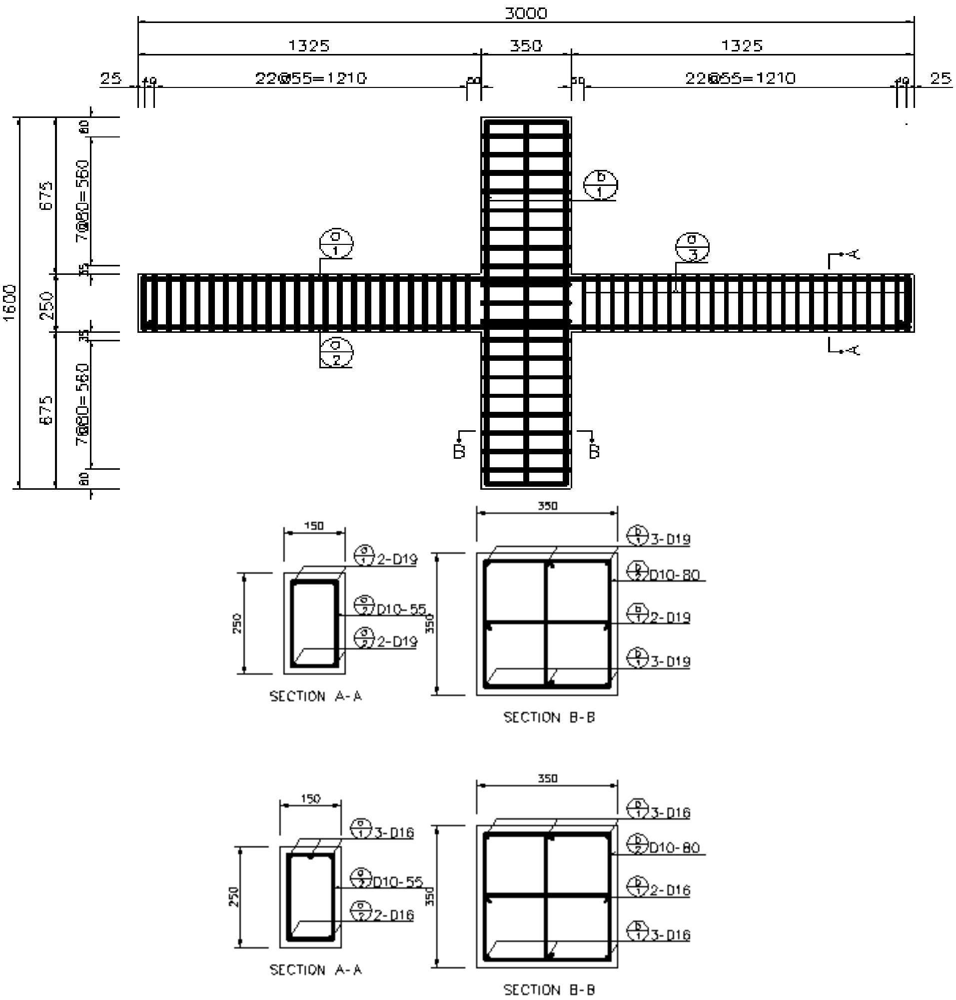


Fig. 1. Specimens' details (Kurniawan et al), (a) Elevation view of the specimen, (b) Beam and column section for SD19 (Top) and SD16 (Bottom)

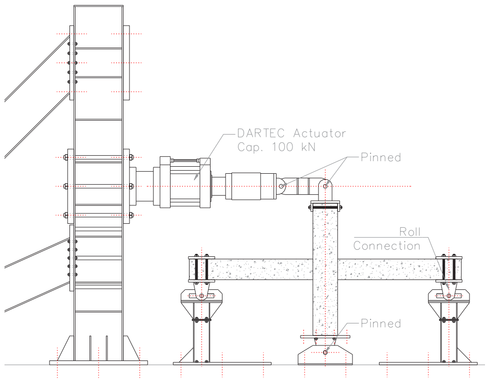


Fig. 2. Specimen Setup for Cyclic Loading

(a)

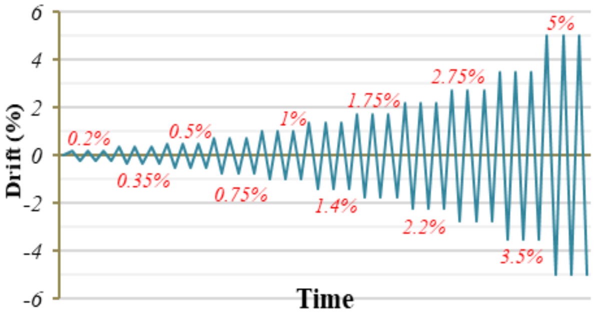
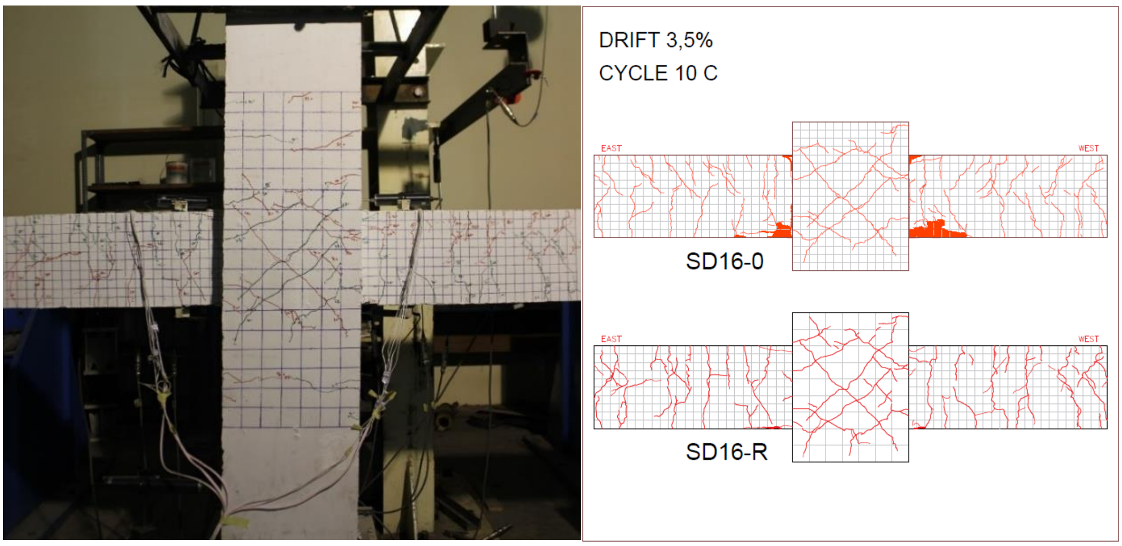


Fig. 3. Module for Cyclic Loading

3 Test Results

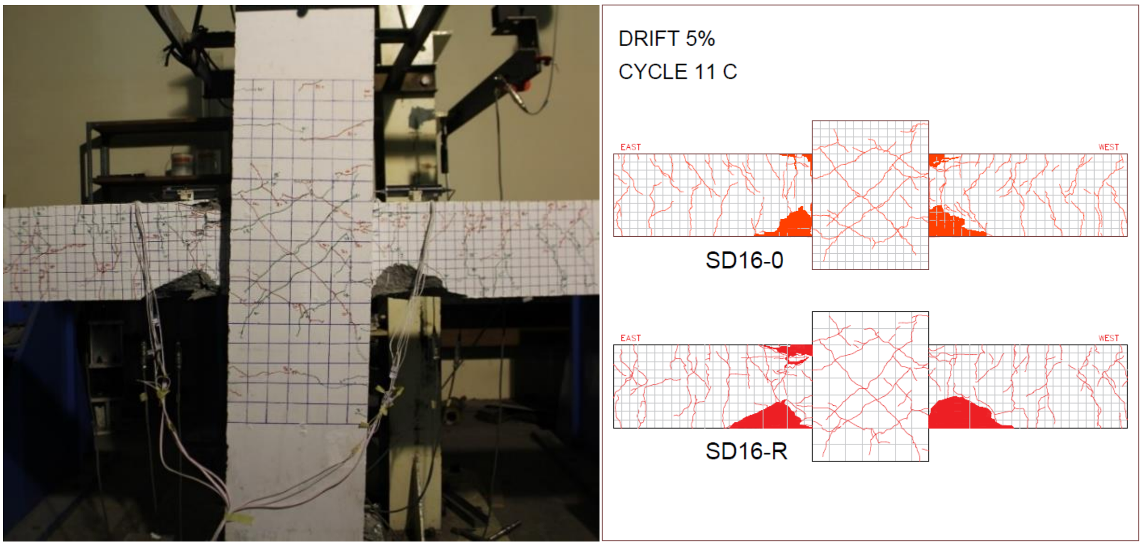
Failure mechanism of both specimens are the same, where at first, vertical cracks form on the bottom and top edge of the beams. Those vertical cracks then followed by the reopening of the diagonal cracks on the column surface, which begin to extend until the beams start to spall, showing the stirrups inside the beams. Both columns have not yet spalled. The damaged specimens can be seen from Fig. 4 to Fig. . Cracks forming on the specimens are then analyzed and compared. As shown in Fig. and Fig. , the repaired specimens show similar maximum lateral force compared to their initial behavior. Moreover, SD16-R's maximum lateral force is shown higher than its initial behavior. However, repaired specimens seem have thinner curves and smaller stiffness compared to the initial specimens. Comparison of maximum lateral forces can be seen in Table 1.

Stiffness of each specimens are then compared at every first cycle (Cycle A) for each drift level. The stiffness value is acquired by measuring gradient of the curve from peak to peak at each cycle. Dissipated energy of each specimens is also compared to see whether the repaired specimens are able to dissipate as much energy as their initial condition. It is measured by calculating the area that is bounded within the hysteretic curve for each cycle and is added cumulatively. Due to limitations in the lab's testing machine, specimens are only tested until they reach drift level of 5% instead of their failure. In this drift level, it can be seen that both specimens' beams have achieved spalling (Fig. and Fig.) while the beams' longitudinal rebars have achieved their yield stress, as can be seen in Fig. and Fig. assuming their yield strain is 0.002.



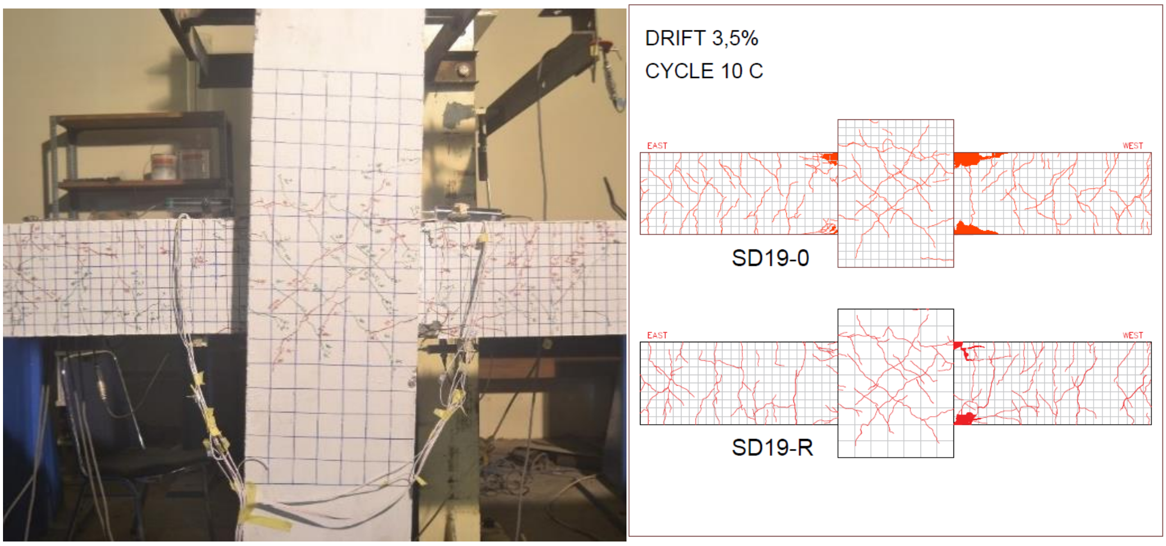
(a) (b)

Fig. 4. (a) Specimen SD16-R at Drift Level 3.5%, (b) Comparison of Crack Patterns for SD16 at Drift Level 3.5%



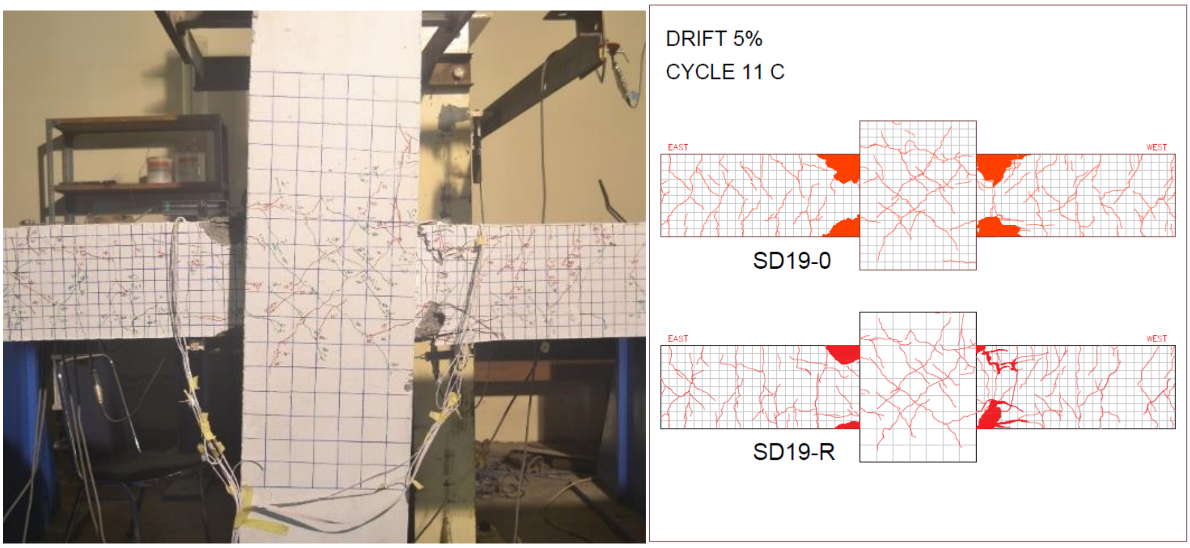
(a) (b)

Fig. 5. (a) Specimen SD16-R at Drift Level 5%, (b) Comparison of Crack Patterns for SD16 at Drift Level 5%



(a) (b)

Fig. 6. (a) Specimen SD19-R at Drift Level 3.5%, (b) Comparison of Crack Patterns for SD19 at Drift Level 3.5%



(a) (b)

Fig. 7. (a) Specimen SD19-R at Drift Level 5%, (b) Comparison of Crack Patterns for SD19 at Drift Level 5%

4 Discussion

Error! Reference source not found. and Fig. show the crack patterns for SD16 at drift level 3.5% and 5% respectively while Fig. 6 and Fig. shows the crack patterns for SD19. As can be seen from **Error! Reference source not found.** to Fig. , cracks are mostly forming vertically on the repaired specimens' beams. This proves that both of the specimens' beams are experiencing flexural failure rather than shear failure, proving that both designs have accomplished stronger shear capacity. Furthermore, it can be seen that both specimens are failing from beam failure, proving that both designs already agree to strong column weak beam principle. It is also can be seen that diagonal cracks forming on both specimens' column is similar to their initial condition, hence proving that the repaired specimens' stiffness is smaller than their initial stiffness because of the cracks in the column that aren't repaired. Lastly, it seems that spalling in both repaired specimens occurred at a later drift level compared to their initial condition. This is due to the repaired specimens' smaller stiffness, causing both specimens to experience smaller load at the same drift level.

Fig. 1 and Fig. present stiffness degradation of both specimens. It can be seen that SD16-R's stiffness, ranging from drift level 0.2% to 3.5% is usually constant at a value of 1.1 kN/mm while SD19-R's stiffness is constant at a value of 1.25 kN/mm until drift level 2.75%. This constant stiffnesses seems caused by reopening of the cracks that are already formed before. Both stiffnesses then start to decrease when the beams start to spall, similar to their initial stiffness degradation.

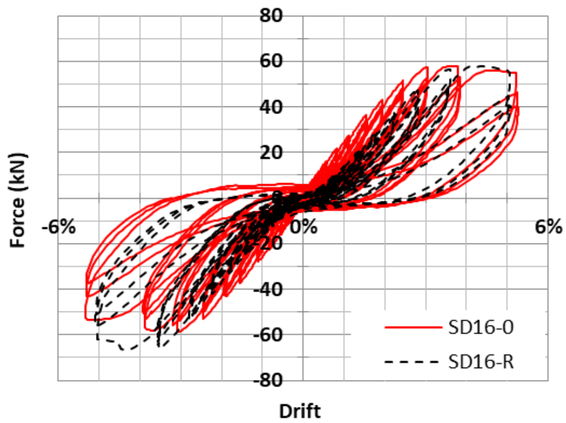


Fig. 8. Hysteretic Curve Comparison of SD16

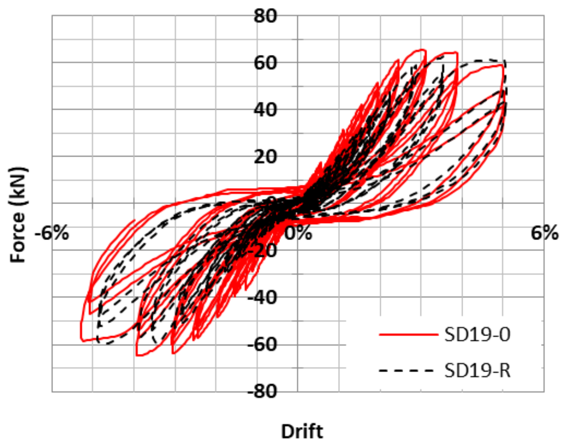


Fig. 9. Hysteretic Curve Comparison of SD19

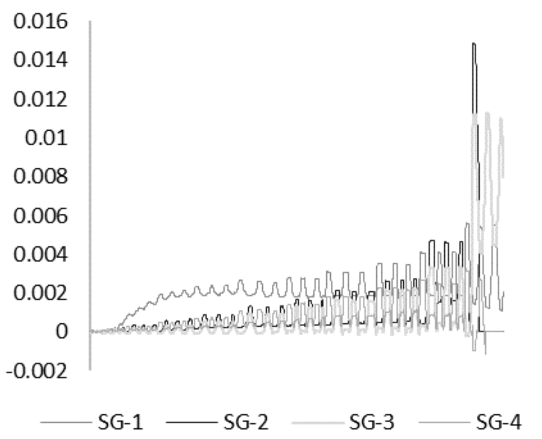


Fig. 10. Strain Diagram for SD16 Beams' Longitudinal Rebars

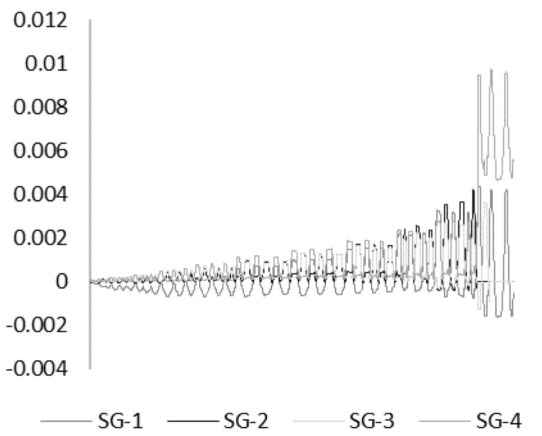


Fig. 11. Strain Diagram for SD19 Beams' Longitudinal Rebars

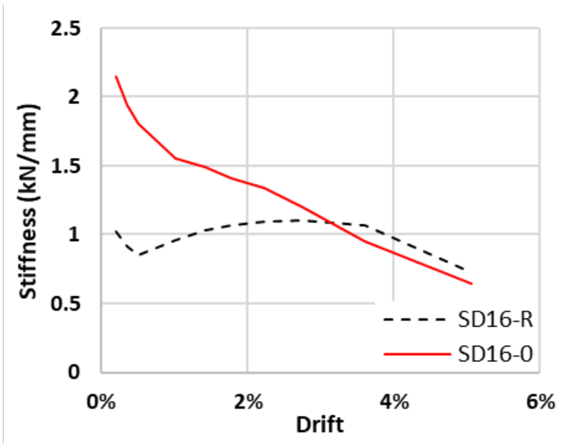


Fig. 1. Cycle-A Peak to Peak Stiffness Comparison for SD16

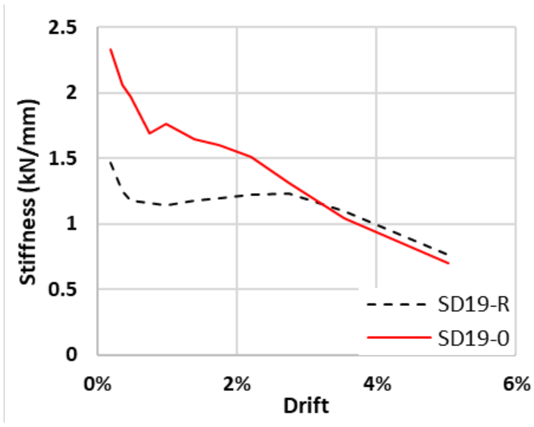


Fig. 13. Cycle-A Peak to Peak Stiffness Comparison for SD19

As it can be seen from Fig.8, Fig. 9, and Table 1, repaired specimens' maximum lateral force is usually lower than its initial condition, ranging from 0.02% to 7.96% lower. However, for SD16-R, its negative maximum lateral force can reach 13.23% higher than its initial condition. These unique behaviors are due to the difference in residual stress and strain inside the beams' reinforcement bars. These residual stress and strain can either cause the bars' yield stress and ultimate stress higher or lower according to their loading history. Higher yield stress and ultimate stress in beams' reinforcement bars can cause beams to have higher moment capacity, therefore making the joint have higher lateral capacity, while lowering yield stress and ultimate stress in the beams' reinforcement bars can cause the beams to have lower flexural capacity.

Table 1. Comparison of Maximum Lateral Forces

Specimen	Maximum Lateral Force (kN)		Difference	
	Positive Loading	Negative Loading	Positive Loading	Negative Loading
SD16-0	57.991	-58.881	-0.02%	13.23%
SD16-R	57.981	-66.673		
SD19-0	65.555	-64.784	-4.80%	-7.96%
SD19-R	62.407	-59.63		

Fig. 4 and Fig. 25 show the comparison of energy dissipated by the specimens. Due to their smaller initial stiffness, both repaired specimens reach their peak lateral force later than their initial condition. This causes them to dissipate less energy than before at 5% drift. While SD16-0 and SD19-0 can dissipate energy for a total of 22,662 kNmm and 25,744 kNmm respectively, their repaired counterparts can only dissipate for a total of 17,082 kNmm for SD16-R and 18,861 kNmm for SD19-R. In another words, the repaired specimens can only dissipate energy for 0.73 to 0.75 of its initial condition using this reparation method.

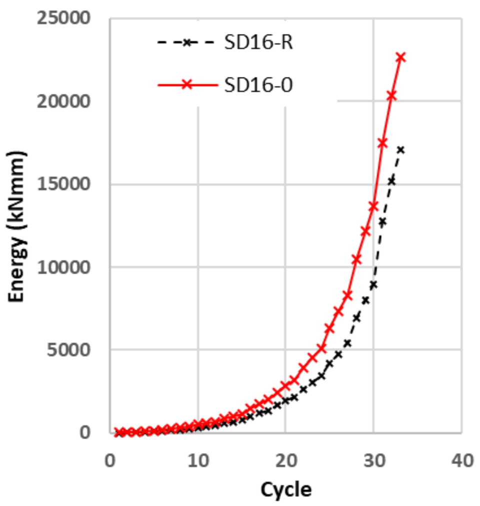


Fig. 14. Energy Dissipation Comparison for SD16

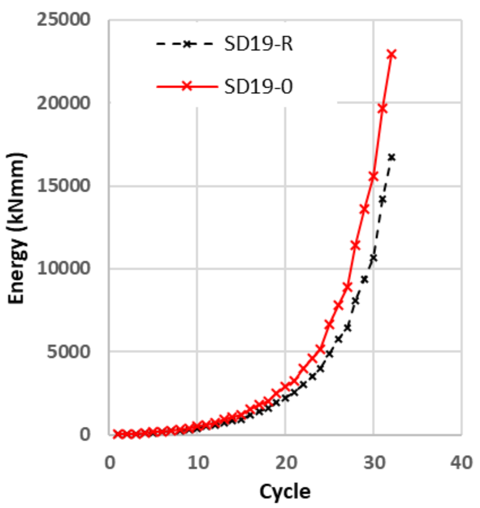


Fig. 25. Energy Dissipation Comparison for SD19

SNI 1726:2012 states the definition of deformability, which is the ratio of an ultimate displacement of a structure against its limit displacement. This definition can be seen in Fig. 36. Further description tells that limit displacement is twice the displacement where the strength of the specimen reaches 0.4 of its maximum strength (0.4 E_{max}), while ultimate displacement is a displacement value where the specimen's lateral force is degrading until 0.8 of its maximum strength (0.8 E_{max}). Due to the actuator's limitation on its maximum displacement, both repaired and initial specimens can only achieve 5% drift, without their strength degrading into 0.8 E_{max} yet. These results are shown and compared in Table 2 and Table 3 for specimen SD16 and SD19 respectively. Due to their lack of stiffness, repaired specimens have larger value of limit displacement, thus making their deformability lower than their initial condition.

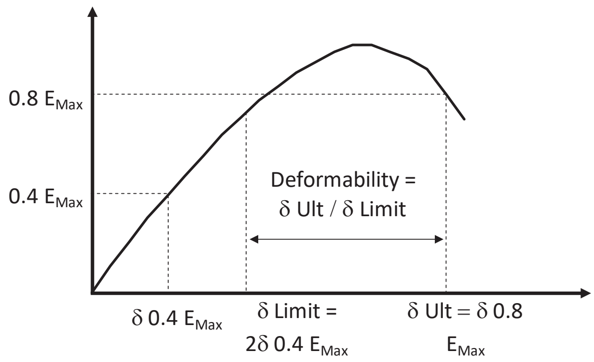


Fig. 36. Deformability According to SNI 1726:2012

Table 2. Deformability Comparison of SD16

Loading Direction	Specimen	Deformability			
		E_{max} (kN)	δ_{lim} (mm)	δ_u (mm)	δ_u/δ_{lim}
Positive	SD16-0	57.991	27.573	83.700	3.036
	SD16-R	57.981	47.806	81.030	1.695
Negative	SD16-0	-58.881	-29.148	-84.330	2.893
	SD16-R	-66.673	-47.838	-80.610	1.685

Table 3. Deformability Comparison of SD19

Loading Direction	Specimen	Deformability			
		E_{max} (kN)	δ_{lim} (mm)	δ_u (mm)	δ_u/δ_{lim}
Positive	SD19-0	65.555	29.434	79.500	2.701
	SD19-R	62.407	38.220	80.580	2.108
Negative	SD19-0	-64.784	-30.544	-80.310	2.629
	SD19-R	-59.630	-45.720	-77.52	1.696

5 Conclusion

Based on the experimental results, it can be concluded that while this repair method can attain similar – or even larger peak lateral capacity due to the reinforcement bars' residual stress, both repaired specimens have lower stiffness compared to their initial condition, thus making this method still ineffective in repairing severely damaged reinforced concrete beam column joints. In order to optimize the repaired specimens' performance, column reparations are also needs to be done to ensure no loss of stiffness. Strengthening of specimens also need to be done such as adding more reinforcement bars or wrapping specimens with FRP to ensure no loss in peak lateral capacity. Other conclusions are as follows:

- The differences of peak lateral forces between repaired specimens (SDXX-R) and their initial condition (SDXX-0) vary from 7.96% lesser to 13.23% more due to the residual stress inside the beams' reinforcement bars.
- The energy dissipated by the repaired specimens is smaller than their initial condition by 25 to 27%.

- Based on SNI 1726:2012 and assuming the ultimate displacement is at 5% drift, repaired specimens have lower deformability than their initial condition.

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