International Journal of GEOMATE, Oct. 2023, Vol. 25, Issue 110, pp.114-122
ISSN: 2186-2982 (P), 2186-2990 (O), Japan, DOI: https://doi.org/10.21660/2023.110.3987
Geotechnique, Construction Materials and Environment

STUDY OF REPAIRED SPUN PILE TO PILE CAP CONNECTIONS CONTAINING FRACTURED REINFORCEMENTS USING FRP

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*Corresponding Author, Received: 19 June 2023, Revised: 20 July 2023, Accepted: 21 July 2023

ABSTRACT: There is a need for a study on the seismic performance of foundations to facilitate the implementation of performance-based design for bottom structures in Indonesia. To address this need, two series of experimental studies were conducted specifically on spun pile-to-pile cap connections, which are commonly used in Indonesia. The objective of the study was to analyze the performance of these connections and their post-repair behavior. Two specimens were tested: one with an empty pile (SPPC01) and another with a concrete-filled pile (SPPC03). Despite the limited confinement of the piles, both specimens exhibited ductile behavior, achieving displacement ductility within the range of 3.0 to 6.0. The test found that the connections were capable of developing the inelastic stage of the prestressed bars. Both specimens experienced irreparable damage due to fractures in the bars. They were repaired using Fiber Reinforced Polymer (FRP) and retested. The specimens showed some differences in their behavior. SPPC01R exhibited rocking with uplift because the connection was merely embedded, while SPPC03R, which had sufficient embedment, experienced FRP fracture. Although the stiffness of the connections could not be fully restored, the repaired specimens demonstrated a strength recovery of 71% and 86% compared to the original specimens and were able to reach 87% of the design strength. Additionally, they exhibited ductile behavior with ductility ranging from 3.43 to 6.9. To conclude, the typical spun pile connections in Indonesia show ductile behavior. For optimal repairability, it is advised to fill the pile with reinforced concrete, particularly in the connection area.

Keywords: Spun pile, Pile cap connections, Fibre-reinforced polymers, Strength recovery, Ductility

1. INTRODUCTION

Indonesia is located in an active seismic region and frequently experiences strong earthquakes. Notable recent earthquakes include the ones in Lombok and Palu in 2018, which were followed by devastating tsunamis and liquefaction. Recent seismic mapping indicates an increasing intensity of earthquakes compared to previous years. However, the design code for the bottom structures still adheres to the elastic concept, requiring piles to be designed to withstand earthquakes without sustaining damage. Consequently, an enormous number of piles are needed to ensure elastic behavior during severe earthquakes. This approach contradicts the government's policy of accelerating nationwide transportation construction. Therefore, Indonesia must transition towards Performance-Based Design (PBD). To expedite this process, several relevant researches are necessary.

Spun pile is a commonly used foundation in Indonesia, especially for bridges, wharves, and medium-rise buildings. It is a circular hollow precast prestressed pile. However, the production of spun piles lacks sufficient transverse reinforcement, as required in ASCE 41-19. This deficiency has not yet become a significant concern due to the adoption of the elastic design concept, ensuring that the pile does not experience any inelastic behavior.

The behavior of spun piles and their connections remains not fully understood, especially within the context of Indonesia. The connection plays a crucial role in transferring loads between the upper and bottom structures. A rigid connection, in particular, can result in maximum curvature, leading to the formation of plastic hinges or damage at this specific section.

To date, no studies have been conducted specifically examining the behavior of the spun pile-to-pile cap connection using the typical connection details employed in Indonesia. Previous studies conducted in the last decade, such as those referenced in [1-4], explored different connection configurations. In Indonesia, the spun pile is connected to the pile cap through extended prestressed bars using a specific length. To achieve the intended flexural strength, the spun pile is filled with reinforced concrete at the connection region. This study reports an experimental investigation of the spun pile-to-pile cap connection, including an examination of repairing.

The term "repair" refers to the process of restoring the performance of a structure that has experienced deterioration due to significant loading, such as an earthquake. While numerous studies have focused on repairing reinforced concrete columns, there is a lack of research specifically addressing pile repair. This is primarily because the construction method for piles is more complex compared to that of upper structures. For embedded piles, the surrounding ground needs to be excavated, making the repair process more challenging. In such cases, a commonly used method for repairing damaged piles is jet grouting, which involves injecting a concrete slurry to reinforce the surrounding soil and the damaged pile itself [5]. However, this method is only effective for light damage, such as spalling or crushing of the concrete cover, and cannot be implemented for severe damage, such as buckling or fracture of the longitudinal rebar.

The column bridges' damage levels are classified according to the NCHRP 440 [6], which divides them into five categories. Repairability is possible for Levels I to IV, provided that the core concrete remains intact and there is no splitting or buckling of the longitudinal reinforcement. Damage level V, on the other hand, requires significant repair efforts or even complete column replacement. Research on the seismic performance of different piles for bridges conducted by [7] proposed a similar five-level classifications of damage for concrete piles based on the AASHTO [8] guidelines.

While damage level V is typically classified as irreparable, there have been several studies focusing on repairing bridge columns at that level of damage. For instance, a study conducted by [9] explored quick repairs using Carbon Reinforced Polymers (CFRP). The research demonstrated that the proposed rapid repair procedure was both practical and effective in restoring strength and displacement capacity. Another repair concept was introduced by [10-11], involving the relocation of the plastic hinge to a higher location in the column. This repair method utilized FRP wrap and carbon fiber (CF) anchors. Similarly, [12] reported the innovation of plastic hinge relocation by using the grouted annular ring to repair the damage. The preceding studies demonstrate that the use of CFRP and carbon fiber successfully restores the strength of bridge columns that have sustained severe damage, including buckling and failure of reinforcement bars.

This paper presents an experimental study focused on the repair of damaged spun piles connected to the pile cap. The findings of this study could be applied to elevated piles commonly employed as foundations for bridge piers or wharves. The research was conducted in two sequential series. The first series examined the original spun pile connections, while the second series evaluated the repaired connections based on the findings from the preceding series. The repair method employed Fiber Reinforced Polymers (FRP).

The research objective was to investigate the seismic performance of the connections between the spun pile and the pile cap, as well as to assess to which these connections could be effectively repaired when experiencing severe damage. The performance evaluation was conducted based on strength, ductility, and energy absorption. To measure the Repairability of the connections, a performance comparison was made between the second series and the first series.

2. RESEARCH SIGNIFICANCE

The research conducted on the spun pile to pile cap connection serves as an important step towards supporting the transition from an elastic design concept to PBD in Indonesia. Since the seismic behavior of the typical spun pile connection details used in Indonesia remains poorly understood, the lack of experimental studies in this area highlights the need for comprehensive research. Understanding the behavior of the connection and its Repairability is essential for implementing PBD.

3. EXPERIMENTS

3.1 Preparation of Original Specimens

To assess the seismic performance of the spun pile connections, two full-scale specimens were prepared. One specimen, SPPC01, was an empty spun pile, while the other, SPPC03, was filled with reinforced concrete. Prior to the experimental phase, a preliminary numerical study utilizing finite element analysis was conducted [13]. This study provided initial insights and served as a reference to estimate the strength of the specimens. The design of the specimens ensured that their strength did not exceed the capacity of the loading frame.

In Indonesia, the most commonly used spun pile has a diameter of 500mm. However, due to restrictions in the capacity of the forklift to move the specimens from the yard into the laboratory, a diameter of 450 mm with a wall thickness of 80 mm was selected. To closely represent real conditions, the piles used in this study were not specifically fabricated. A section measuring 220 mm in length was cut from the middle of the pile. The diameter of the spiral was 40 mm, with a spacing of 120 mm. The pile was made of concrete with a strength of 57 MPa. It was reinforced with 10 prestressed concrete (PC) bars with a diameter of 7.1 mm. The volumetric ratio of the reinforcement was 0.113%. This value only accounted for 13% of the minimum requirement specified in ASCE 7-16, indicating that the amount was significantly lower. Fig. 1 provides the Detailed Engineering Design (DED) of the specimens.

The pile cap used in the study had dimensions of 1700 x 1200 x 700 mm. The size was designed to have ten anchorages with a space of 500 mm to attach the specimens to the strong floor. The cap was constructed using concrete with a strength of 35 MPa. The reinforcement employed was D19-150 bars.

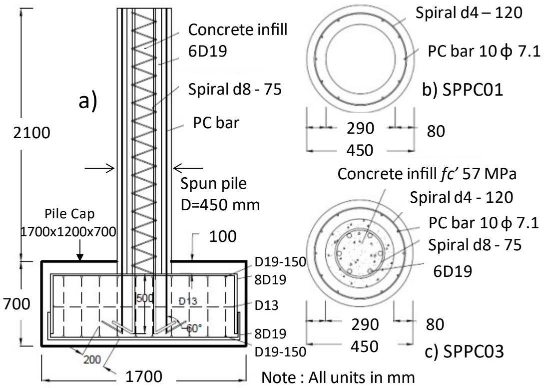

Fig.1 DED of the original specimens (a) cut section of pile cap connection; (b) cross-section of spun pile SPPC01; (c) cross-section of spun pile SPPC03.

The pile was embedded into the pile cap to a depth of 100 mm. The PC bars were anchored to the pile cap where the required embedment length specified in article 25.4.2.3 of Indonesia's design code, SNI 2847:2019, was 620 mm. To reduce the overall depth of the pile cap, the anchor length was designed with a straight portion of 500 mm and a bent portion of 200 mm, angled at 30 degrees, as depicted in Fig 1a. The properties of the steel reinforcement used in the specimens are provided in Table 1.

Table 1 The Steel Properties

The Rebars	f_y (MPa)	f_u (MPa)
Prestressed Bar ϕ7.1	1275	1562
Spiral of Pile d4	390	703
Rebar D19	400	570
Spiral of Concrete infill d8	240	370

3.2 Preparation of Repaired Specimens

3.2.1 The Damage

Both specimens were tested until failure. The details of the test setup can be found in Section 3.3. However, it is worth noting that SPPC01 did not reach the targeted drift of 3.5% during the test and was halted at a drift of 2.75%.

The crack patterns observed on the spun pile and pile cap are depicted in Fig 2. In the case of SPPC01, cracks on the spun pile extended to approximately 400 mm from the surface of the pile cap. For SPPC03, the cracks reached a distance of about 650 mm. A few shear crack, which was indicated as inclined crack, was found near the connection region of SPPC03, as can be seen in Fig 2d. In general, flexural cracks were predominantly observed for both specimens, as clearly shown in Fig. 2a and 2b. Additionally, the spalling of concrete was observed at the connection region of the pile cap in both specimens. Cracks were also detected on the concrete cover of the pile cap. These cracks originated from the connection region and propagated to a radius of approximately 150 mm to 180 mm from the face of the pile.

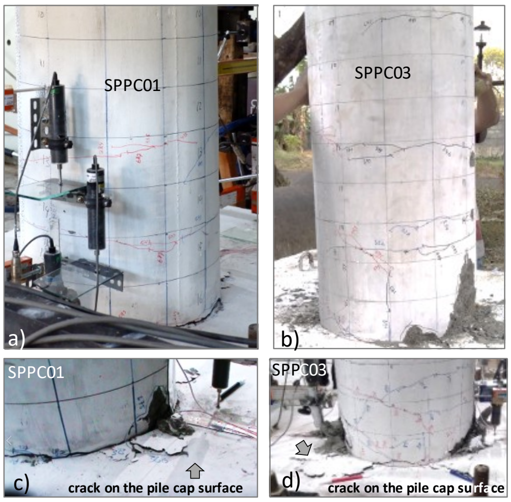

Fig. 2 The cracks patterns (a-b) on the spun pile; (c,d) on the pile cap surface

During the testing, it was observed that the PC bars had fractured. These fractures became visible after removing the loose concrete from the surface of the spun pile. The location measured from the surface of the pile cap is illustrated in Fig 3. This observation indicates that the embedded length of the PC bars in the pile cap was sufficient to develop the ultimate strength of the bars. Furthermore, no buckling was detected on the bars despite the limited confinement provided by the spun pile.

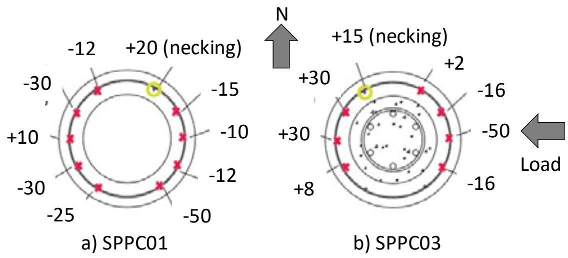

Fig. 3 Location of the fractured prestressed bars

Based on the test results, it was found that the specimens SPPC01 and SPPC03 experienced a loss in capacity of 49% and 39%, respectively. According to the previous studies [6-8], both specimens can be classified as having "significant damage but not collapse," as evidenced by the fracture of the PC bars. Hence, it is suggested that either replacement with new piles or significant repair efforts are necessary.

3.2.2 Repairing Process

The objective of the repair was to restore the strength and ductility of the specimens. For this purpose, FRP plates (Sika Carbodur S-512) were utilized to replace the PC bars, while FRP wrap (Sika 231C), was employed to confine the repaired region. These FRP materials are widely utilized for retrofitting or repairing structural components, such as beams and columns, in Indonesia. The properties of the FRP materials are outlined in Table 2.

Table 2 The properties of FRP

Material	*E* and f_u (MPa)	Elongation
FRP Plate S-512 50 mm width and 1.2 mm-thick	165000 2900	1.8%
FRP Wrap 231 C 0.129 mm-thick	200000 3850	1.91%

The repaired specimens were named SPPC01R and SPPC03R, with the addition of "R" to signify the repair. The design of the repair process was carried out according to Indonesia's design code, SNI 8971:2021, which references ACI 440.2R-17. In order to achieve similar strength to the original specimens, a total of 14 FRP plates were used. Additionally, two layers of FRP wrap were applied to confine the repaired zone. The volumetric ratio of FRP wraps was 0.50%, which is slightly lower than the minimum requirement of 0.54%.

Figure 4 depicts the sequence of the repairing process. A tripod was utilized to hold the pile in place, ensuring its verticality during the repair process. In the case of SPPC01R, which was initially an empty spun pile, a filling process of 650 mm height of concrete was introduced to reinforce the crack present on the inner side of the spun pile. The next step involved removing any loose concrete, including all visible cracks on the pile's cover, along with an additional 100 mm to ensure that any invisible cracks were also eliminated. The total height of chipping on the pile amounted to 850 mm from the pile end. The extent of chipping on the concrete cover of the pile cap varied according to the observed crack patterns. Subsequently, the FRP plates were glued to the spun pile by using an adhesive agent, Sikadur-30, followed by grouting the concrete cover of the spun pile using instant concrete with a strength of *fc*' 50 MPa. After seven days, the repair concrete was wrapped with the FRP. Finally, the concrete cover of the pile cap was grouted using a similar concrete mixture, incorporating small aggregates 10-15 mm in size.

Figure 5 illustrates the DED of the repaired specimens. As shown, the FRP plate was positioned adjacent to the PC bars, extending for a total length of 650 mm until the pile end. The FRP wrap covered a depth of 750 mm in the repaired region, which was 100 mm longer than the plate. Both the FRP plate and wrap were embedded into the pile cap by 100 mm. Due to the asymmetrical location of the fracture bar in SPPC03, the cross-section of SPPC03R also became asymmetric as a result of the repair process.

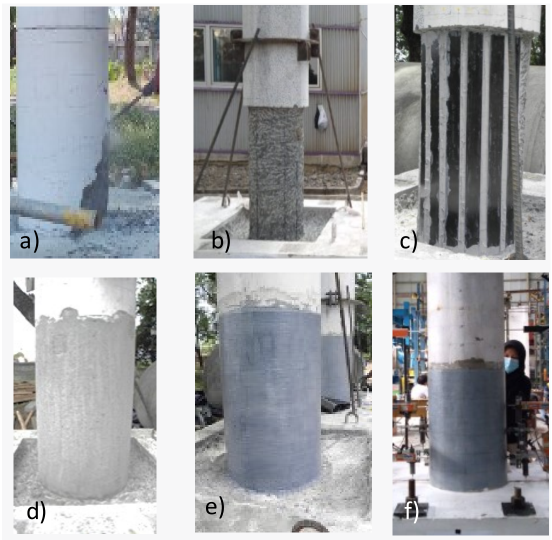

Fig. 4 The repairing: (a-b) removing the loose concrete (c) applying FRP plate (d) grouting the spun pile (e) applying FRP wrap (f) ready

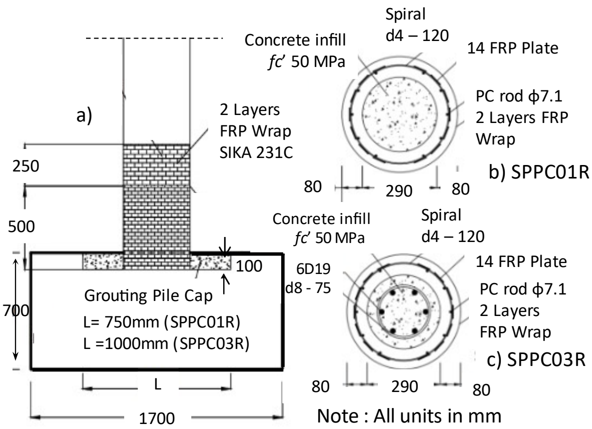

Fig. 5 DED of repaired specimens; (a) side view; (b-c) cross section of spun pile SPPC01R; (c) cross-section of spun pile SPPC03R

3.3 Test Setup

The specimens from both experimental series were tested until failure using a cyclic horizontal loading protocol as specified by ACI 437-07, where the targeted drift was 3.5%. Prior to applying the cyclic horizontal load, a constant vertical load of

+500 kN was fully applied, which was equivalent to ten percent of the axial strength of the spun pile. The test setup, as depicted in Fig 6, involved attaching the specimen to a strong floor and securing it with ten anchoring bolts to prevent any movement. To prevent slippage of the pile cap, two equal horizontal forces were applied at the bottom of the pile cap. A steel plate measuring 500x500 mm with a thickness of 12 mm was placed on top of the pile to transfer the force from the actuator. Additionally, a vertical jack was attached to a sliding frame to ensure the vertical alignment of the loading.

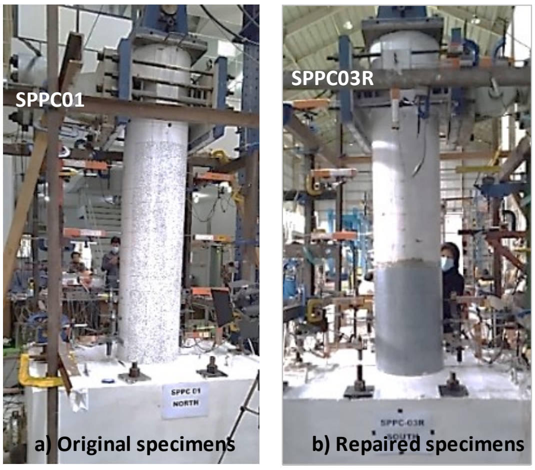

Fig. 6 The Experimental Setup; a) original specimen; b) repaired specimen

Five transducers attached to each side of the spun pile were used to measure the horizontal displacement of the pile. They were located at 150 mm, 450 mm, 900 mm, 1350 mm and 1800 mm from the pile cap surface. Meanwhile, four transducers were employed to measure the vertical displacement and the rotation of the piles which were located at depths of 150 mm and 300 mm. Additionally, the movement of the pile cap was monitored using two vertical and horizontal transducers.

Strain gauges were installed on the PC bars and the reinforcement bars (D19) of the spun pile. The gauges were placed at two different locations: 120 mm and 170 mm below the surface of the pile cap, parallel to the horizontal load direction. Similarly, strain gauges were also installed on the D19 reinforcement bars parallel to the gauges on the PC bars. After the first series test, fractures were observed in the PC bars that had attached gauges. It is important to note that gauges were not installed on the FRP. Therefore, for the repaired specimens, the only gauges were on the longitudinal reinforcement, D19 bars.

4. TEST OBSERVATION

As previously mentioned, SPPC01 had an empty spun pile connected to the pile cap using PC bars. However, during the first test, all bars were fractured. In the subsequent repair process, the bars were replaced with FRP plates, which were not anchored to the pile cap. Consequently, the connection between the pile and the cap was simply embedded, with the pile penetrating only 100 mm into the pile cap. As a result, SPPC01R exhibited increased flexibility compared to SPPC01, enabling it to reach a drift of 3.5% during testing.

The failure mode of SPPC01R is shown in Fig 7. Since the pile was not rigidly connected to the pile cap, an uplift of the spun pile was observed during the test. Therefore, there was no damage found on the pile. Vertical cracks appeared on the surface of the pile cap due to uniaxial tension in the concrete caused by uplift forces. The occurrence of these cracks was influenced by the adhesion of the concrete materials [14].

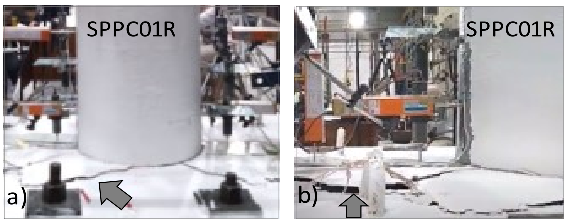

Fig 7 a) The crack pattern of SPPC01R; b) severe vertical cracks on the pile cap surface

In the case of SPPC03, although seven PC bars were fractured during the first test, two bars and six D19 remained intact to connect the pile to the pile cap. Therefore, the connection was sufficient to develop the ultimate strength of the longitudinal reinforcement similar to SPPC03. Figure 8 and 9 illustrates the failure mode of SPPC03R. Six FRP plates were fractured, and their locations from the pile cap surface are shown in Fig 8. Localized large cracks appeared on the spun pile at the same location as the fractured plates. These failure modes indicate that FRP plates and wrap were strongly attached to the pile, and the friction between the wrapped and the surrounding concrete was optimal. Severe vertical cracks were also detected on the pile cap surface at the interface between the original and repaired concrete, as shown in Fig 9d.

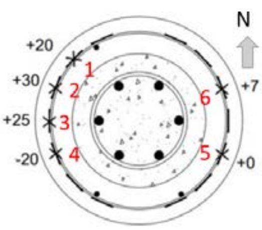

Fig. 8 Location of fractured plate of SPPC03R

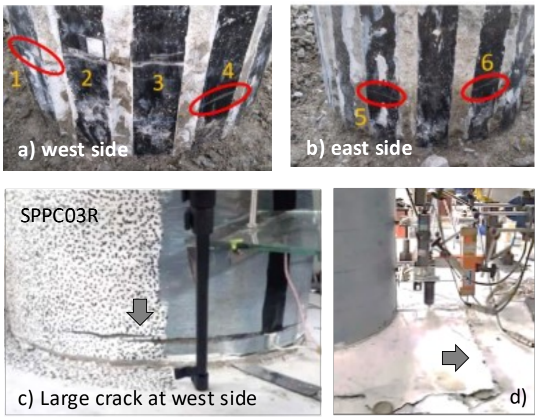

Fig 9. Failure Modes of SPPC03R, (a-b) fractured plates; (c) large crack on spun pile; d) vertical crack on pile cap surface

5. TEST RESULTS AND DISCUSSION

5.1 Hysteretic Response

The hysteretic curves of both series are presented in Fig 10 and Fig 11. In the case of SPPC01, the strength increased until a drift of 1.35%, while for SPPC01R, the strength continued to increase until a drift of 2.0%. Fig. 12 shows the hysteretic curves of the last cycle for both specimens. The curves are associated with a phenomenon called "rocking" [15], which refers to the condition where the pile becomes detached from the pile cap. In the case of SPPC01R, it occurred at a drift of 3.5%. Similarly, the original specimen, SPPC01, also experienced rocking when the PC bars fractured consecutively, at a drift of 2.75%. Uplift, the vertical displacement or movement of the pile due to rocking, was 12.2 mm for SPPC01 and 14.6 mm for SPPC01R.

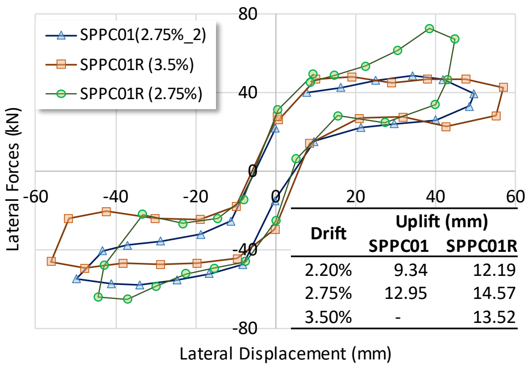

Fig 12. The comparison of the last hysteretic curve of SPPC01 and SPPC01R

The hysteretic curve of SPPC03R in Fig. 11 exhibits an asymmetrical failure mode, which aligns with the previously described asymmetric cross-section and the asymmetric failure mode illustrated in Figure 8a. It is observed that the connection exhibited greater strength during pull loading compared to the other direction. The strength of SPPC03R almost reached a similar level as SPPC03 due to the fracture of four plates. However, on the other side, only two FRP plates fractured, resulting in a maximum strength of only 68.5% of SPPC03.

The cumulative dissipated energy, which is measured as the area enclosed by the hysteretic curve of each cycle, provides insights into the capacity of the specimens to absorb the energy. The total amount of cumulative dissipated energy for both series is shown in Fig. 10 and Fig. 11. It is observed that the repaired specimens have slightly lower energy dissipation compared to the original specimens at the same drift level. However, due to their ability to reach longer drift levels, the repaired specimens ultimately achieve higher overall dissipated energy. SPPC01R exhibited a dissipated energy of 33.46 kNm, whereas SPPC01 had 28 kNm. SPPC03R achieved a dissipated energy of 72 kNm, which was 36% higher than SPPC03.

5.2 Strength

For a clear comparison, Fig. 13 and Fig. 14 show the skeleton curves of both series. The effectiveness of the repair process in restoring the strength of the original specimens is evaluated using the strength recovery (SR) parameter. SR is calculated as the ratio of the maximum strength of the repaired specimens to the original specimens.

Table 3 provides a strength comparison between the original and the repaired specimens. The strength of the original specimens exceeded the design strength by a ratio of 1.2. It indicates that the connection was sufficient for developing the inelastic behavior of the PC bars and allowing the spun pile to achieve its maximum strength.

Table 3. The Summary of Strength

Specimen	Yield Moment (kNm)	**M**aximum Moment (kNm)	**D**esign Moment (kNm)	Ratio **M/D**
SPPC01	150.24	196.56	153.92	1.28
SPPC01R	123.03	139.16	159.68	0.87
SPPC03	237.78	276.81	224.72	1.23
SPPC03R	201.01	237.78	271.76	0.87

SPPC01R, which had a different connection detail resulting in a different failure mode, achieved a strength recovery (SR) of 71%, indicating that it was not able to fully restore the strength of SPPC01.

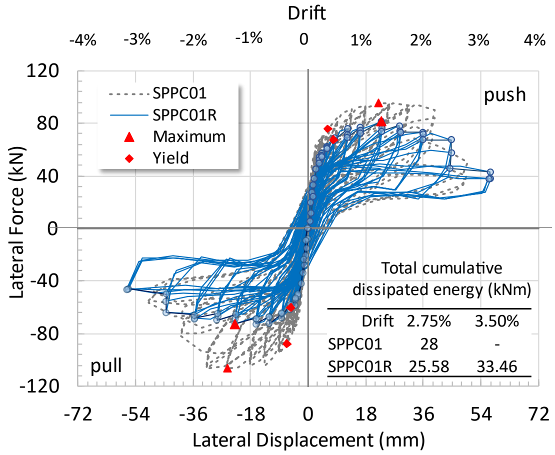

Fig 10. The hysteretic curve of SPPC01/R

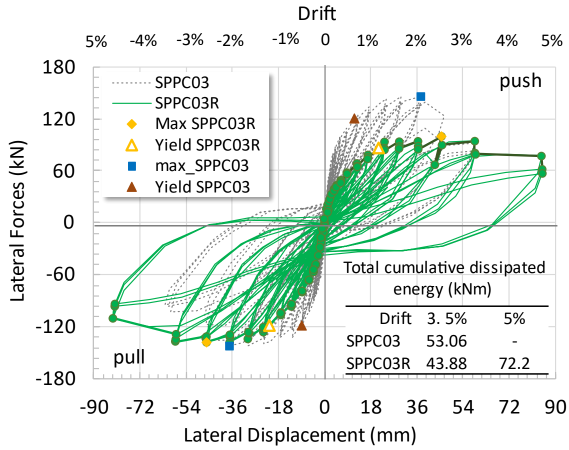

Fig 11. The hysteretic curve of SPPC03/R

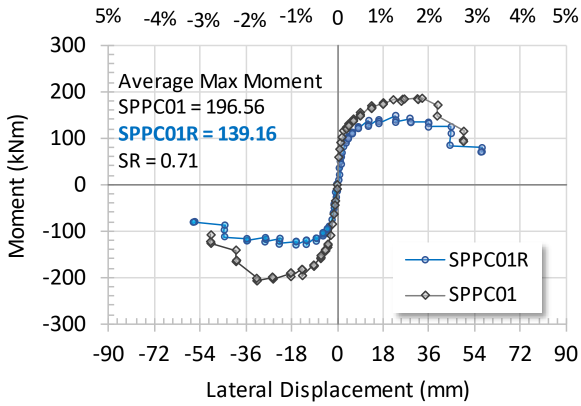

Fig. 13 The envelope curves of SPPC01/R

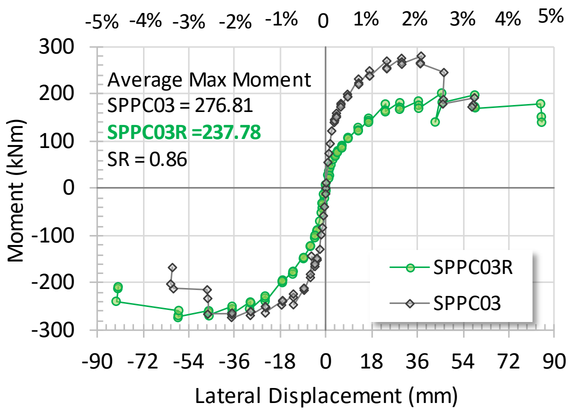

Fig.14. The envelope curves of SPPC03/R

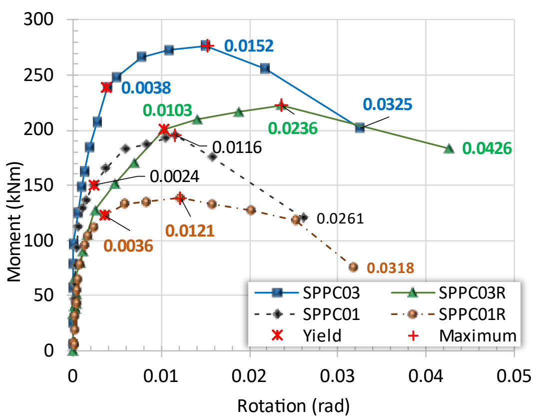

Fig. 15. The comparison of Moment – Rotation Curves of both series

On the other hand, SPPC03R achieved a higher SR of 83%. When comparing the maximum moment obtained in the experiments to the design moment from the P-M interaction, SPPC01R and SPPC03R reached 87% of the strength. This revealed that they almost reached the design moment.

The moment-rotation curves presented in Fig. 15 provides a comparison of all specimens, showing their rotational behavior. Since repaired specimens were less rigid than the original ones, the yield and maximum capacity were observed at larger drift values. Notably, the rotational capacity of the repaired specimens was found to be better than that of the original ones. Specifically, SPPC03R exhibited a rotation of 0.0426 radians, which was 31% higher than the rotation achieved by SPPC03. Similarly, SPPC01R attained a rotation of 0.0318 radians, representing a 47.2% increase compared to SPPC01.

5.3 Initial Stiffness

Table 4 reveals the comparison of initial stiffness (*Ki*) among the specimens. It is calculated as the ratio of the peak lateral load of each cycle to the corresponding lateral displacement. It serves as an indicator of the rigidity of the connections. Based on the table, it can be observed that the repair was unable to fully restore the stiffness of the original specimens. This can be attributed to the loss of embedment of the PC bars in the repair specimens.

They exhibited lower average *Ki* values compared to the original specimens.

Table 4 Initial Stiffness and Ductility

	SPPC01	SPPC01R	SPPC03	SPPC03R
K_i (kN/m)	24.97	19.26	29.46	13.93
Ductility 1	3.69	3.58	3.37	2.15
Ductility 2	6.04	6.92	4.95	3.43

By comparing *Ki* of the repaired specimens to the original specimen, it appears that SPPC01R, with the addition of concrete infill in the spun pile, was able to achieve 77% of the initial stiffness of the original specimen. This indicates that the concrete infill contributed to the rigidity of the spun pile connection. Conversely, SPPC03R exhibited an initial stiffness of only 47% compared to SPPC03.

5.4 Ductility

Ductility is an important parameter in assessing the seismic performance of structures, as it measures the ability of a structure to undergo large displacements without experiencing a significant loss in strength. Ductility is determined as the ratio of ultimate displacement (*du*) to yield displacement (*dy*). While ductility may not be suitable for structures that exhibit rocking behavior, it is used to compare the effectiveness of the repair to the original specimens.

Figure 16 illustrates an example calculation of the ductility of SPPC03R. Previous researchers [1-4] have adopted two different definitions of ultimate displacement. These definitions include the displacement at peak load (du_1) and at post-peak with 15% strength degradation (du_2). The yield displacement is determined based on the secant stiffness [1]. Consequently, there are two possible ductility values corresponding to du_1 and du_2, respectively. Additional assessment of the ductility of the initial test is elaborated upon in [16].

Table 4 provides a comparison of the ductility values obtained from both tests. SPPC01R exhibits ductility values, D#1 and D#2, of 3.58 and 6.92, corresponding to du_1 and du_2, respectively. Meanwhile, SPPC01 demonstrates ductility values of 3.69 and 6.04. According to the definition by du_1, SPPC01R appears to be less ductile than SPPC01. However, upon comparing the skeleton curve in Fig 13 and the moment-rotation curve in Fig 15, it becomes evident that the post-peak behavior of SPPC01R is more steady compared to SPPC01. As a result, the definitions based on du_2 closely align with the ductility exhibited by SPPC01R, indicating that it is indeed more ductile than SPPC01.

SPPC03R exhibits lower ductility compared to SPPC03. This is due to the fact that yield occurs at a larger displacement of 21 mm for SPPC03R, whereas SPPC03 yielded at 10 mm. Consequently, the calculated ductility of SPPC03R is 3.43, which is lower than the ductility value of SPPC03, which is 4.95. However, it is important to note that SPPC03R demonstrates improved rotational capacity and greater lateral displacement than SPPC03, as indicated in Figures 10, 12, and 15. According to AASHTO [8], the displacement ductility of the bottom structure is limited to 4.0, and thus, all specimens are considered ductile.

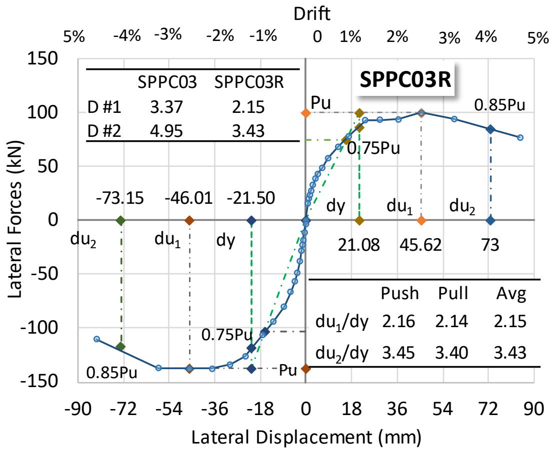

Fig.16 The calculation of ductility of SPPC03R

6. CONCLUSION

The experimental study found that the connection between the spun pile and the pile cap is sufficient to achieve the required strength of the spun piles. The strength of the connections exceeds the design strength of the spun pile, with ratios of 1.23 for SPPC03 and 1.28 for SPPC01. It is noteworthy that despite the limited transverse reinforcement in the spun pile, the connection exhibits ductile behavior. This is evidenced by the ductility values of the two specimens, which are measured as 3.43 for SPPC03R and 6.04 for SPPC01R.

Due to the difference in connection details, the full-strength restoration of the repaired specimens could not be achieved. SPPC01R and SPPC03R managed to recover 71% and 86% of the original specimens' strength, respectively. Additionally, they were able to reach 87% of the design strength. The repaired specimens exhibited slightly lower displacement ductility compared to the original specimens. The displacement ductility for SPPC01R and SPPC03R were measured as 3.43 and 6.9, respectively. In general, the repaired specimens demonstrated ductile behaviour.

In conclusion, the typical spun pile connection commonly used in Indonesia exhibited ductile behavior and proved to be reparable, even in situations where significant damage was present.

The addition of reinforced concrete filling in the connection region played an important role in maintaining the adequacy of the connection during severe earthquakes. The presence of the reinforced concrete filling significantly impacted the effectiveness of the repair, ensuring the preservation of the structural integrity and post-repair performance of the spun pile-to-pile cap connection.

7. ACKNOWLEDGMENTS

The research is supported by the Indonesia Ministry of Research and Universitas Indonesia through a Doctoral Research Grant, contract number NKB-101/UN2.RST/HKP.05.00/2022 and part of Applied Research Grant, contract number NKB-254/UN2.RST/ HKP.05.00/2021 with PT Wijaya Karya Beton, Tbk.

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