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Experimental and Numerical Analysis of Spun Pile-to-Pile Cap Connection with Reinforced Concrete Infill under Cyclic Loading

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Abstract-Numerical study was conducted by using ABAQUS software to investigate two issues on spun pile connections. The issues are usage of non-shrinkage concrete infill cast inside hollow of pile and ductility. An experimental test was carried out to investigate the effect of two different concrete infill types which were common concrete fc' 35 MPa and non-shrinkage concrete fc' 54 MPa. A finite element model was validated against experimental test. The results were compared in terms of hysteresis curve, ductility, and performance level. Ductility is one of the important parameters to describe performance of spun pile connections under seismic load. There are different approaches to calculate ductility and this leads to four different ductility values. Hence, ductility of spun pile with concrete infill is in the range of 3.2 to 4.8. Behavior of spun pile with non-shrinkage concrete infill is slightly improved although the results were almost similar.

Keywords- *Ductility, Non-Shrinkage Concrete, Pile Cap, Spun Pile*

I. INTRODUCTION

Design of prestressed pile-to-pile cap connection on seismic zone in Indonesia is guided by SNI 1726-2019. For fixed connection, prestressed bar from the pile should be embedded into pile cap with a specified length. In order to accommodate higher curvature on the connection region, mild rebar can be added. Concrete infill is also given inside the hollow of spun pile with approximately one meter depth. Figure 1 shows the anchorage of concrete infill reinforcement to the pile cap.

Study of spun pile and its connection has not attracted such a considerable amount of research in Indonesia since the design code is still based on the elastic concept. Lateral displacement of pile is restricted to 12 mm for the earthquake design load and 25 mm for severe earthquakes for single-pile with free-head condition [13]. In the other words, the bottom structure is still designed based on elastic condition and damage is not allowed due to difficulty of the repair process. In fact, many foundations were damaged after severe earthquake [6], [10]. However, the increase of seismic demand in Indonesia leads to a larger size of foundation in order to meet the requirements of strength and displacement. This is diametrical with financial aspect of structure development. Procurement cost of earthquake-resistant structures should be optimal, especially in the era of infrastructure development.

Performance based design has been widely used on the upper structure design which allows structure to behave inelastic during severe earthquake. The design concept has been adopted on the substructure in several countries based on results of previous research of spun pile-to-pile cap connections [17], [3], [14], [16]. Ductility is one of the critical parameters to ensure that the structure can behave inelastic during seismic. It describes the capability of structure to behave in inelastic phase. The structure should undergo large amplitude cyclic deformations without a substantial reduction in strength. To achieve that performance, sufficient transverse reinforcement is needed to prevent shear failure and to ensure that concrete has enough confinement during the inelastic phase. However, confinement of spun piles in Indonesia has not reached the ACI 318-19 requirements. For example, a 450 mm diameter spun pile confined by 4 mm spiral with a pitch of 120 mm has a volumetric ratio of 0.11%, which is only 10% of the minimum requirement. Inadequate confinement results in low ductility which is not a problem in elastic design concept. Nevertheless, sooner or later, Indonesia should move to performance-based design since the seismic demand based on the latest seismic risk maps tend to increase. As a transition to performance-based design of the substructure, more research on pile-to-pile cap connections based on common practice in Indonesia is needed. The study has been conducted on low confinement of spun pile under seismic load [7], [8]. It is found that the ductility of spun pile with a diameter of 400 mm was less than 5 which is categorized as low ductility according to Japan's code.

There are various methods in calculating ductility, specifically in determining ultimate and yield displacement [12], which leads to different ductility values. Therefore, it is necessary to explore the true meaning of ductility to avoid any misleading interpretations. Two spun pile connections were studied experimentally. The spun pile was not a special order but taken randomly from an existing production. Application of strain gauges on the specimens were not accurate, hence further study was performed numerically based on FE analysis to investigate the stress and strain development of the concrete and steel in the specimens. The FE models were validated based on the experimental results. ASCE 61-14 was used to assess the performance level based on the strain limit.

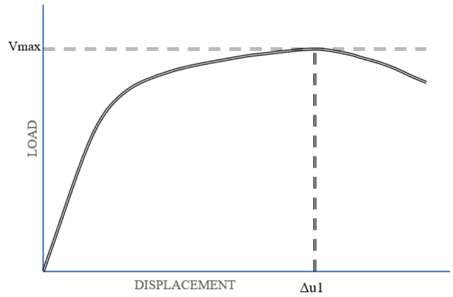
Figure 1. Spun pile-to-pile cap connection

Application of non-shrinkage concrete on the spun pile-to-pile cap connection has been widely used in many key projects that the government is currently engaged in. Although this application is not required formally in any codes, it is intriguing to discover the behavior of non-shrinkage concrete on spun pile-to-pile cap connection. Based on previous numerical studies, the inner side of spun pile also suffered tensile stress and leads to cracking. Hence the capacity of spun pile reduced, and crack propagation might occur in location where repair is not possible. Over a long period of time, gaps will form between the spun pile and concrete infill, resulting in slippage. Therefore, an experimental study was conducted on a 450 mm diameter spun pile to determine the impact of non-shrinkage concrete.

II. LITERATURE REVIEW

Research on spun piles has been carried out in the last few decades. According to Muguruma [11], the lesson that can be learned from the Kobe earthquake is that the spun pile foundation must be designed as a ductile member. In a study conducted by Irawan [8], the presence of infilling concrete can increase the ductility of the spun pile. Hence the seismic performance of spun pile also increases and thus the spun pile can be used in areas with moderate earthquake risk. Bang [3] studied about the effect of modified transverse reinforcement in the concrete infill that could increase ductility of the structure and avoid shear failure. Moreover, spun pile without steel anchorage against the pile cap had undergone brittle failure. Yang and Wang [16] conducted a study about the effect of various steel anchorage against the pile cap. The result indicated that the presence of continuous concrete infill and the straight steel anchorage develop a good behavior.

According to Park [12], ductility is the ability of a structure to resist deformation in the inelastic phase without any reduction in strength. Structural components with a ductility value of more than four met the criteria for high ductility demand based on FEMA 356. Displacement ductility is defined as the ratio of the ultimate displacement to yield displacement. There are two alternative definitions for ultimate displacement (Figure 2), as well as the yield displacement (Figure 3). Therefore, there are 4 possible values of ductility. Those methods have been adopted by previous studies [14], [7], [15]



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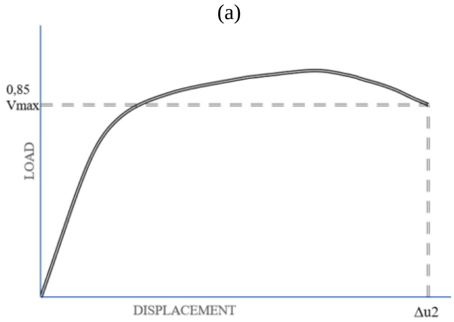

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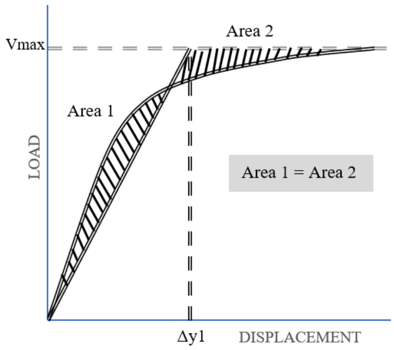
Figure 2. Definitions of ultimate displacement based on a) peak load and b) significant load capacity after peak load [14] [15]



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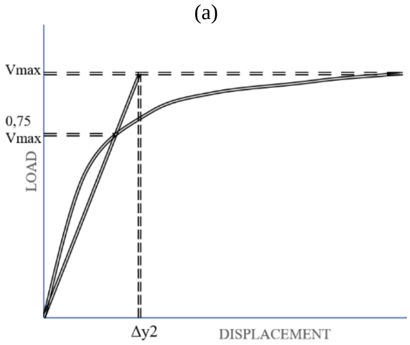

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Figure 3. Definitions of yield displacement based on (a) equivalent elasto-plastic energy absorption, and (b) reduced stiffness equivalent elasto-plastic yield [14] [7]

III. RESEARCH METHODOLOGY

Two specimens designated as SPPC 02 and SPPC 03 were prepared for testing under cyclic loading. Both specimens had a hollow section with outer 450 mm diameter and a wall thickness of 80 mm. SPPC 02 has 34,2 MPa concrete infill as well as the concrete of the pile cap. Meanwhile SPPC 03 was filled with non-shrinkage concrete with fc' 54,3 MPa. The spun pile was embedded at 100 mm depth. There were 10 PC wires with 7,1 mm diameter and six rebar with 19 mm diameter as the anchorage against the pile cap. Figure 4 shows the details of the specimens.

TABLE I. MATERIAL PROPERTIES OF CONCRETE

Component	fc'(MPa)	
	SPPC 02	SPPC 03
Spun pile	57,8	
Pile cap	34,2	36,5
Concrete Infill	34,2	54,3 (non-shrinkage)

TABLE II. MATERIAL PROPERTIES OF STEEL REINFORCEMENT

Component		fy (MPa)	fu (MPa)
Spun pile	Longitudinal PC Wire (10⌀7,1 mm)	1224	1440
	Spiral (⌀4-120)	390	703
Concrete Infill	Longitudinal (6D19)	400	570
	Spiral (⌀8-75)	240	370
Pile cap	Long & Lat Reinforcement (D19-150)	400	570
	Shrinkage Reinforcement (D13)	400	570

Table 1 and Table 2 show the material properties of concrete and steel reinforcement, respectively. An illustration of experimental set-up is shown in Figure 6. Pile cap was anchored to the strong floor with a total of 10 anchors. Due to the gap between pile cap hole and steel anchor, it was necessary to apply a lateral load to the pile cap to ensure that there is no horizontal displacement of the pile cap. The amount of axial force applied to the pile cap was 25 tonf. Hydraulic jack was installed at a height of 1800 mm from the top surface of the pile cap. A modified steel pipe was attached to the spun pile in order to distribute the load evenly following the shape of pile surface. The axial load applied to the spun pile was about 10% of the compressive strength of the spun pile, which was 50 tonf. Lateral load was applied to the pile according to the loading protocol in Figure 5.

Figure 7 shows the assembled components in ABAQUS software. Transverse spun pile reinforcement were modeled in a spiral form. Concrete stress-strain curve using Mander model and steel stress-strain curve using a true stress-strain curve that has considered the necking effect. Interaction between solid elements was assumed to be hard-contact with friction of 0.1, except for the interaction between the pile and the steel plate was assumed to have no relative body motion. There were two types of loss-of-prestress modeled in the PC wire: lump sum and loss of prestress due to the cutting of pile at a distance of 1 diameter from the cut end of pile.

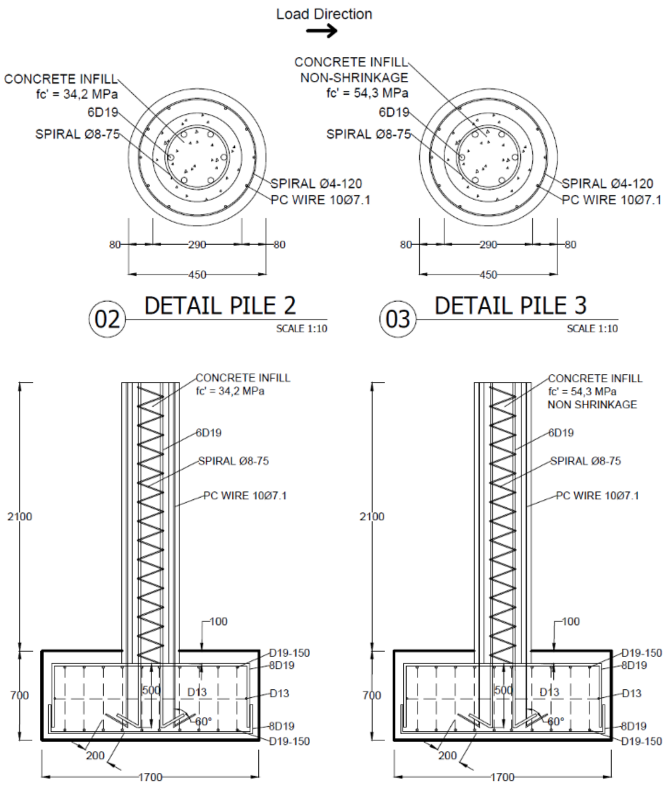


Figure 4. Details of SPPC 02 and SPPC 03 specimen

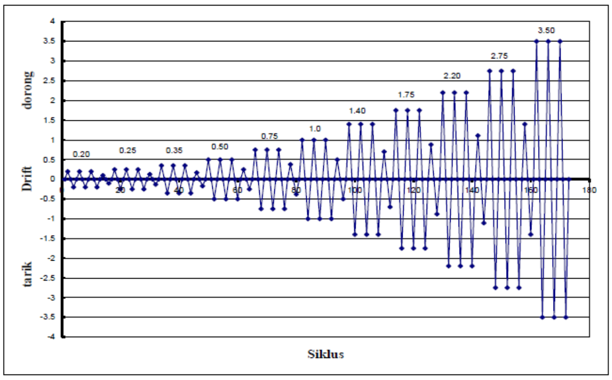
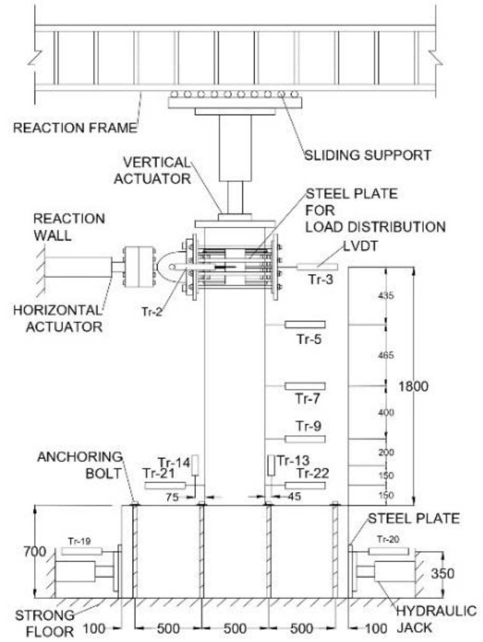


Figure 5. Experimental loading protocol

IV. RESULT AND DISCUSSION

A. *Effect of Non-Shrinkage Concrete*

Figure 8 shows the force-displacement curve of the specimens. Maximum displacement and load on both specimens were at the same drift of 2.2% with approximately the same value, except for pull direction of loading on the specimen SPPC 02, the load was slightly different with the value of 15,39 tf. Force-displacement curve obtained from FE analysis is shown in Figure 9. Visually, both experimental and numerical results exhibited the pinching behavior. Pinching in the reinforced concrete occurs due to crack opening in one side, while the opposite side had the crack closing, then creating the partial stiffness recovery effect. Other causes are shear cracks strain penetration of the longitudinal reinforcement into the pile cap.



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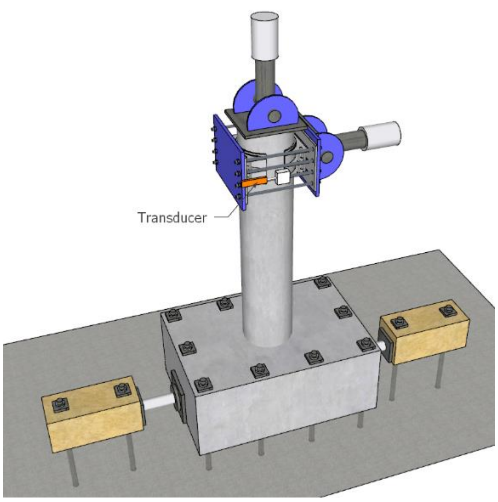

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Figure 6. Experimental set-up

Ultimate displacement and peak load resulting from the modelling showed slightly different values from the experimental results. However, the modelling results showed that SPPC 03 specimen had a higher flexural capacity than SPPC 02 specimen due to the additional compressive strength of the concrete infill.

Seven PC wires fractured at the same spot for both specimens but occurred at the different elevations, as shown in Figure 10, while the SPPC 03 specimen had an additional one necking PC wire. Failure that occurs under the top of pile cap elevation indicates that strain penetration has occurred indeed.

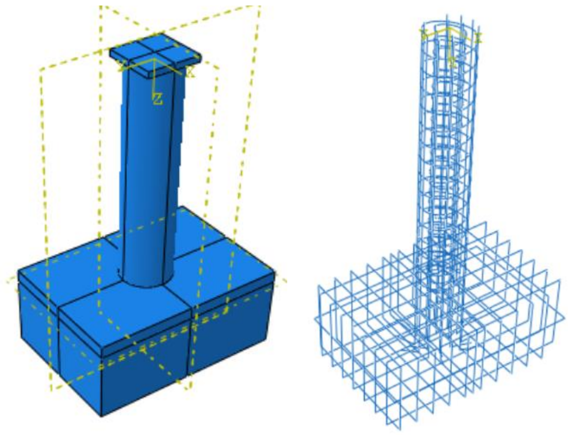
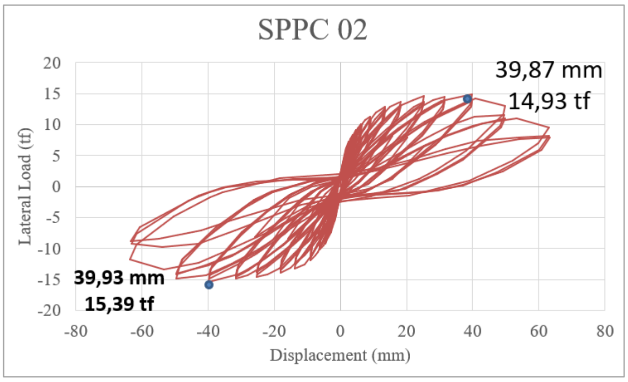


Figure 7. Solid and wire element modelling in ABAQUS software



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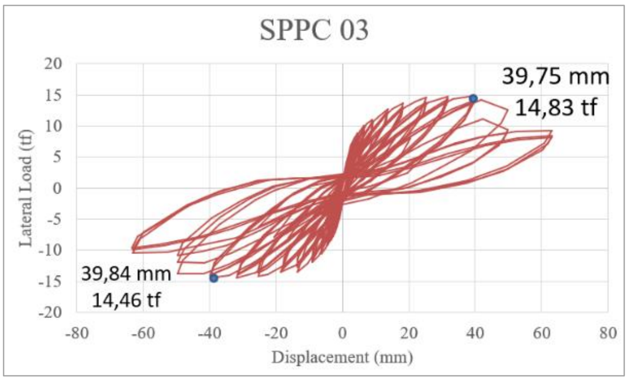

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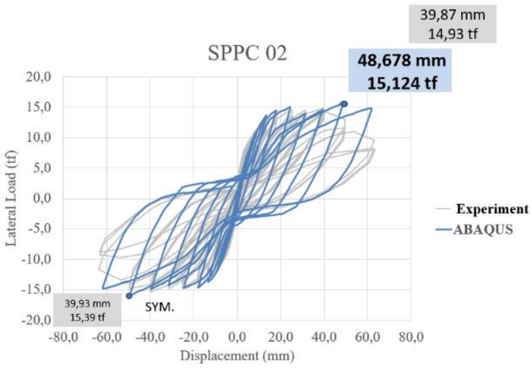
Figure 8. Hysteretic curve of the experimental results of (a) SPPC 02 specimen and (b) SPPC 03 specimen



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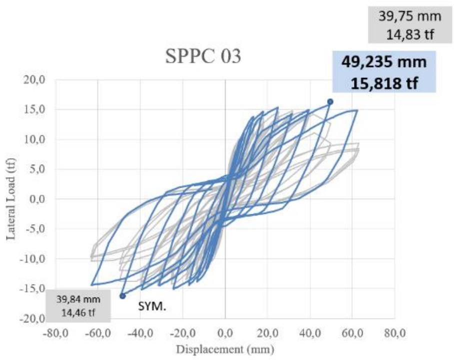

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Figure 9. Comparison between ABAQUS and experimental results of (a) SPPC 02 specimen and (b) SPPC 03 specimen

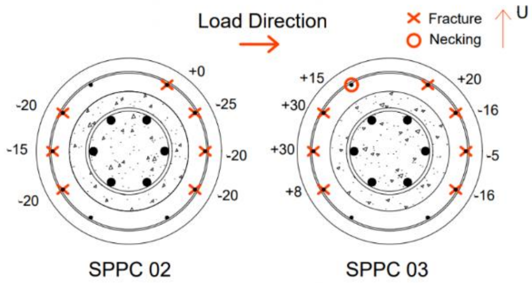


Figure 10. Failure mode of the longitudinal prestressed wire

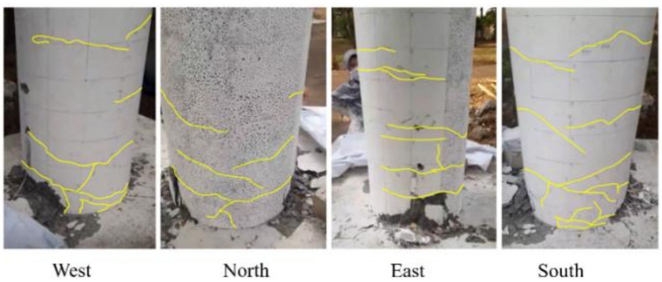


Figure 11. Crack pattern of SPPC 02 specimen

Figure 12. Crack pattern of SPPC 03 specimen

Figure 11 and Figure 12 exhibited the dominance of flexural failure over shear failure, while some shear cracks were still occurring on the spun pile. Crack pattern at the spun pile-to-pile cap connection indicates the yield penetration of reinforcement into the pile cap. Both shear cracks and yield penetration contribute to the pinching effect on hysteretic curve.

The calculation results also show that the ratio of Mu/ϕMn is greater than ratio of Vu/ϕVn, indicating the dominance of flexural failure over shear failure. Table 3 shows the experimental results for each specimen. Plastic hinge length for both specimens was about 18 cm from the pile cap surface based on the measurement.

However, both specimens exhibited the same behavior; hence non-shrink concrete only slightly improves the behavior of spun pile. This is presumably due to the concrete infill on both specimens had not experienced shrinkage because the testing was carried out in the first month after casting. Therefore, it is better to store the specimens for several months before testing in advance to determine the effect of shrinkage because there is a possibility of a gap between the spun pile and the filler concrete in the long term.

B. *Ductility*

Table 4 showed displacement ductility results from the experimental data using 4 different methods of calculation. Ductility of the non-shrinkage concrete specimen is more significant because the yielding occurred earlier so that the value of yield displacement is smaller. Ductility value of SPPC 02 specimen is relatively similar between the push and pull loading, rather than SPPC 03 specimen.

According to FEMA 356, ductility values more than or equal to 4 are included in the category of high ductility demand. By using the 3rd or 4th method, both specimens have reached the requirement of high ductility demand. However, when using the 1st or 2nd method, both specimens are below the high ductility demand requirement. Therefore, different methods of calculating ductility can be misused so that an agreement is needed in determining the ultimate displacement and yield displacement values. Ultimate displacement of building tends to exceed the displacement at the peak load, considering that building is still able to withstand post-ultimate loads. Therefore, the value of $\Delta u2$ is preferable.

TABLE III. TEST RESULTS OF EACH SPECIMEN

Specimen	Max Drift (%)	Maximum Lateral Load (tf)		Initial Crack of Spun Pile	Initial Crack of Pile Cap	Failure Mode	Vu/ϕVn	Mu/ϕMn
		Push	Pull	Drift (%)	Drift (%)			
SPPC 02	3,5	14,93	15,39	0,35	0,2	7 fracture	0,60	0,55
SPPC 03	3,5	14,83	14,46	0,5	0,2	7 fracture, 1 necking	1,37	1,37

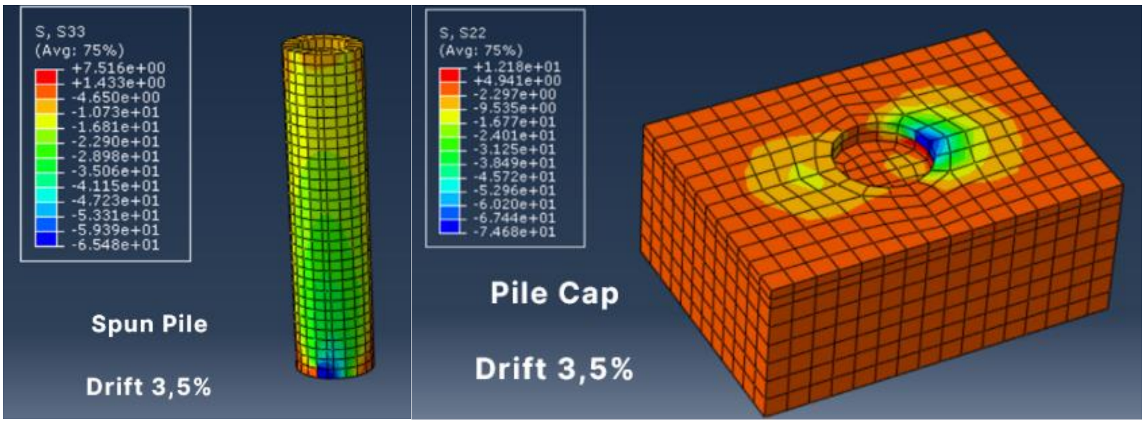
TABLE IV. EXPERIMENTAL DUCTILITY RESULTS OF EACH SPECIMEN

Methods	Ductility SPPC 02			Ductility SPPC 03		
	Push	Pull	Average	Push	Pull	Average
Method 1 (Wang, 2014): $\Delta u_1/\Delta y_1$	3.393	3.003	3.186	3.277	3.999	3.602
Method 2 (Irawan, 2016): $\Delta u_1/\Delta y_2$	3.444	3.480	3.462	3.237	4.142	3.635
Method 3 (Wang, 2019): $\Delta u_2/\Delta y_1$	4.339	4.325	4.331	4.109	5.581	4.773
Method 4 (Irawan, 2016; Wang, 2019): $\Delta u_2/\Delta y_2$	4.404	5.012	4.706	4.059	5.781	4.815

C. Performance Level

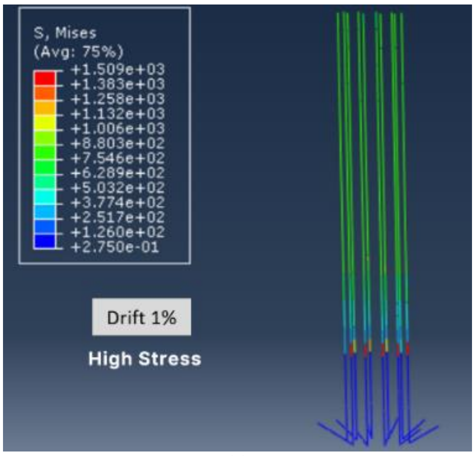
Numerical study using ABAQUS software was carried out to determine stress and strain that occurs in concrete and reinforcement components. Figure 13 shows the location of the plastic hinges on the spun pile and pile cap were in accordance with the experimental results, based on the stress contour that occurs.

Figure 14b and Figure 14c show the stress contour of PC wire and spiral reinforcement, respectively. PC wire suffered high stress at 1% drift ratio, specifically on the spun pile-to-pile cap connection area. At the same place, the entire PC wires had fractured at 3,5% drift ratio, as shown in Figure 14b that the stress value had become 0. This is in accordance with the strain graph shown in Figure 15a where the PC wire suffered high stress at 1% drift ratio and failed at 25 mm displacement. Meanwhile, the stress that occurred in spiral reinforcement had exceeded the yield stress but still below the ultimate stress, as shown Figure 14c. The compressive stress in the spun pile concrete has exceeded its compressive strength, as shown in Figure 15b.



(a) (b)

Figure 13. Stress contour of a) spun pile and b) pile cap at maximum drift ratio



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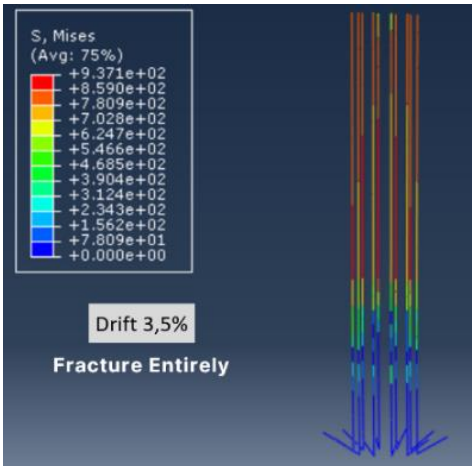

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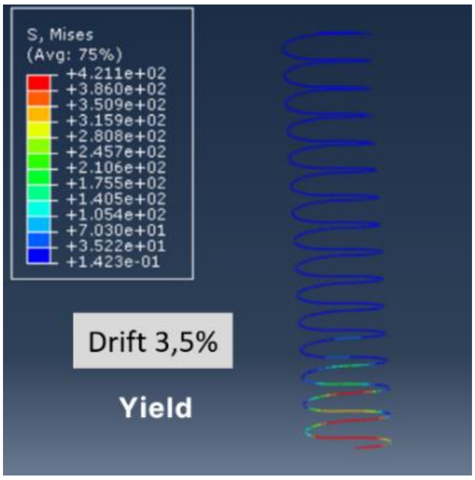
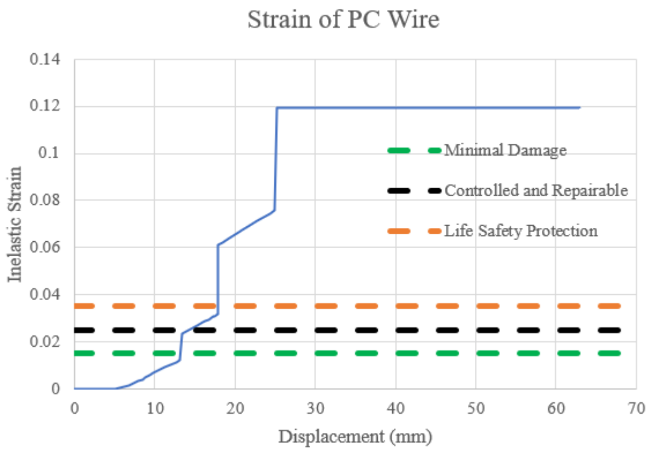


(c)

Figure 14. Stress contours of (a) PC Wire at 1% drift ratio, (b) PC Wire at maximum drift ratio, and (c) spiral reinforcement

Based on ASCE 61-14 in section 3.9, performance level criteria of foundation should be evaluated based on the strain limit. The strain limits are plotted against the strain graph that occurs in the ABAQUS modelling as shown in Figure 15. There are 3 performance level criteria based on its limiting strain; minimal damage, controlled and repairable, and life safety protection. Determination of performance level refers to the limit strain of PC wire because PC wire is more likely to fail than concrete and the process of repairing concrete is relatively easier when compared to PC wire. The PC wire had reached performance level of life safety protection at the displacement of 18 mm.



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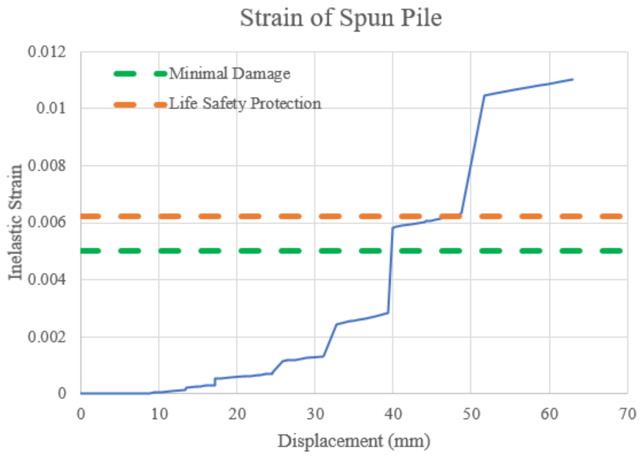

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Figure 15. Comparison between a) PC Wire and b) spun pile strain development

V. CONCLUSION

Application of non-shrinkage concrete in the spun pile-to-pile cap connection only slightly improves the behavior of spun pile. This is presumably due to the concrete infill on both specimens had not experienced shrinkage because the testing was carried out in the first month after casting. Based on the strain limit of the PC wire, it is necessary to restrict the maximum displacement to around 20 mm in order to avoid the fracture of PC wires. Both specimens have reached the requirement of high ductility demand using particular methods of calculating ductility.

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