Increasing the Displacement Ductility Factor of Spun Pile Using Concrete Infill

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Abstract— **Displacement ductility is one of the parameters used to measure the seismic performance of a structure. This study experimentally determines the increase in displacement ductility of the spun pile with 400 mm of outer diameter and 100 mm of wall thickness using concrete infill cast inside the hollow of the pile. The spun pile and concrete infill's concrete compressive strength was 54.4 MPa and 33.0 MPa, respectively. Loading was conducted with constant axial and reversed lateral flexural loads. A total of six samples were tested with different axial loads of 392 kN (0.08fc'Ag) for S-DB-1, S-DB-2, S-DB-5, and 784 kN (0.16fc'Ag) for S-DB-3, S-DB-4, S-DB-6 with the reverse flexure load applied in the middle span of the pile. The results showed spun piles with concrete infill could resist the flexural load combined with axial loads until the displacement ductility 5.8 for $P_0 = 0.08fc'Ag$, and 3.7 for $P_0 = 0.16fc'Ag$, respectively. Compared with the ordinary spun piles, which had a hollow section, the presence of concrete infill due to the presence of the concrete infill the displacement ductility increased by 18% when loaded with 0.08f'cAg and 42% at 0.16f'cAg of axial loads. In conclusion, according to seismic codes, displacement ductility evaluation showed that tested piles for plastic concept design applications are appropriate for moderate seismic risks category state under axial loads of 0.08fc'Ag. The increasing of the axial load becomes 0.16fc'Ag decreasing the displacement ductility to become less than 4, applicable for low seismic risks category state.**

*Keywords*— **spun pile; displacement ductility; concrete infill.**

I. INTRODUCTION

The analysis showed that severe damages occurred on the upper and substructure or foundation buildings in Japan after an earthquake. Similarly, spun pile foundation and damaged piles were depicted from excavation during post-earthquake. For instance, after the Miyagiken-Oki (1978) and Hyogoken-nanbu (Kobe) (1995) earthquakes, some spun pile foundation suffered severe damage, as shown in Fig. 1. The failures were caused by the flexure, shear, compression, or a combination of these forces [1], [2].

Besides, piles were embedded in the ground and also used as freestanding columns constructing marine-related structures such as bridges. Budek and Priestly [3] illustrated the moment pattern on the pile shaft due to lateral load as shown in Fig. 2. According to the head condition, there are two types of pile structures, i.e., free head and fixed head condition. The maximum moment potentially occurs on the

pile shaft section which is embedded in the soil for both head conditions. At the same time, the additional maximum moment occurs on the pile section under the pile head in a fixed condition. The pile section that is emerging above the soil was freely deflected without soil resistance [3].

This research primarily determines the increase in displacement ductility of the spun pile using concrete infill with constant axial and reversed lateral flexural loads. The concrete infill was applied to prevent the inwards crushing of the compression concrete that caused the hollow prestressed pile to undergo a brittle failure, as predicted by previous studies [4]. The wall explosion inside the face occurred on a hollow pile [5] with greater confinement of the concrete core needed to provide greater ductility in terms of bending capacity [6, 7]. Concrete infilling as the core of the pile section contribute to increasing the flexural strength with the ductility factor discussed in this study [8]. This research also determined the axial load effects on the pile

flexural performance with a further analysis of suitable seismic risk designation according to the seismic codes. This study is focused on a spun pile that applicated as a freestanding column above the soil. Therefore, the confinement effect due to soil is ignored. Flexural and axial loading was subjected to the specimens without including the influence of the shear force.
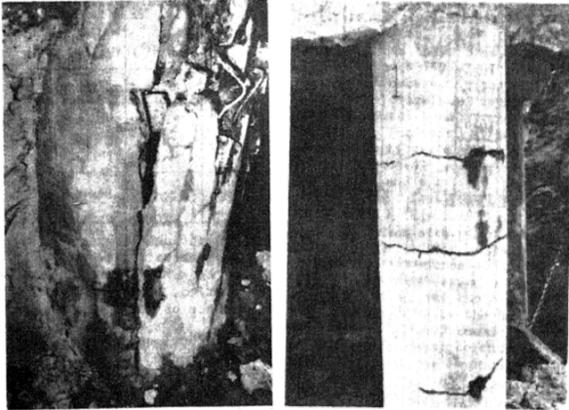
(a). Miyagiken-Oki earthquake 1978
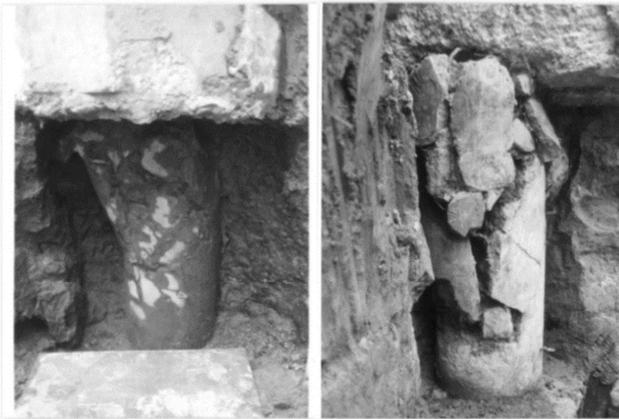
(b) Hyogoken-nanbu (Kobe) earthquake 1995

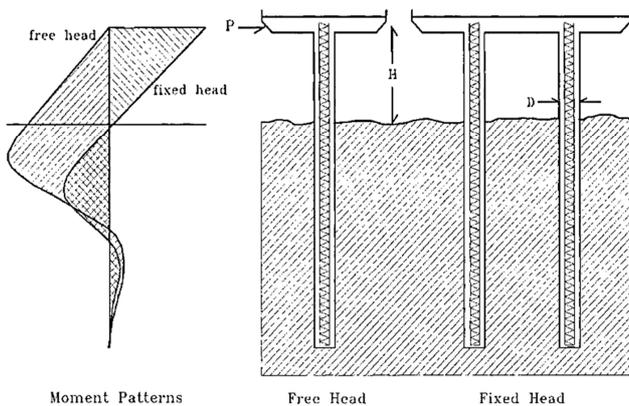Fig. 1 Spun pile damage after the earthquake [1],[2]

Fig. 2 Moment patterns in free and fixed-head piles [3]

Ikeda et al. [9] carried out research to determine damages to the spun pile after earthquakes. The specimens were hollow piles with outer diameters of 400 mm and a wall thickness of 70 mm. The average compressive strength of concrete of the pile was 87.4 MPa. This study aimed to observe the effect of transverse reinforcement ratio and the presence of non-prestressed longitudinal steel. Piles tested by one point lateral load at midspan (unidirectional and cyclic loading) as simply supported with a span of 2.6 m without axial load. The conclusions were adequate core confinement provided by close spacing of transverse reinforcement produces a ductile hollow pile. Piles using non-prestressed longitudinal reinforcement, i.e., deformed bars and unstressed tendons, also had a ductile behavior reaching μ 13 due to unidirectional lateral loading and ductility factor μ 8 under cyclic lateral loading [9].

In Muguruma's study [10], spun piles with high strength concrete, i.e., 95-106 MPa were tested using one point unidirectional lateral midspan without axial load. The pile section prestress was 10 MPa. The pile test configuration was simply supported with a span of 5 m. Piles had an outer diameter of 400 mm, and a wall thickness of 150 mm, twice thicker than Ikeda et al. [9]. The transverse reinforcement using high strength wire in diameters of 5, 6, and 7 mm with a volumetric ratio from none to ρ_t 0.03. The yield strength of wire was 1,000 MPa. Furthermore, the study evaluated the maximum uniform elongation of the prestressing bars at 2%-5.13%. The result showed that the spun pile flexural ductility was proportional to the elongation of the longitudinal steel. High confinement of concrete core provided by sufficient volumetric ratio of transverse reinforcement as confining steel caused crushing damage did not occur on the concrete core section, but the tensile capacity of the longitudinal bars was exceeded on large curvature. Due to longitudinal bar fractures, the mode of failure, such as occurred in this study, is not desirable. The elongation of the prestressing bar should be regarded as its ultimate tensile strain capacity of the pile section [10].

Zahn et al. [11] researched to examine the flexural strength and ductility due to cyclic lateral load and constant axial load by testing six specimens of circular hollow reinforced concrete columns without confining the inner face. All columns had 400 mm outside diameter varied by wall thickness (t). Specimens 1 and 2 had thickness (t) of 94 mm tested under an axial load of 0.08fc'Ag and 0.40fc'Ag, respectively. Specimens 3 and 4 had thickness (t) of 75 mm tested under an axial load of 0.10fc'Ag and 0.22fc'Ag. Meanwhile, specimens 5 and 6 had thickness (t) of 55 mm tested under an axial load of 0.12fc'Ag. In addition, the compressive strength of concrete at 28 days was 29.6 MPa for specimens 1 through 4 and 27.3 MPa for 5 and 6. The transverse reinforcement of specimens 1, 3, and 4 was 1.13%, while 2, 4, and 6 was 1.36%. The results showed that the expected higher ductility of hollow columns is obtained on the column under low levels of axial load, moderate longitudinal reinforcement ratio, and a wall thickness of the pile section not less than 0.15 of the section diameter. However, the level of confining steel ratio did not significantly affect the ductility capacity of hollow columns [11].

Hoshikuma carried out a research and Priestly [5] associated with the flexural performance of circular columns with a hollow section with 1524 mm and 1244 outer and 1244 inner diameters, thereby leading to a 140 mm wall thickness (t/D = 0.092). A single layer of longitudinal and

transverse reinforcement is applied to the column. One of reinforcement was longitudinal with sufficient transverse reinforcement ρ_s, which equals 0.22%. The testing carried out under a 0.13fc'Ag axial load combined with a cyclic lateral load. The test results showed that the columns suddenly failed when concrete on the wall inside face was crushed. It was also found that the plastic hinge region confinement-induced transverse strain did not reach the yield strain before failure. These facts suggest that the limited lateral pressure is induced in the concrete shell. Therefore, the inside face concrete is subjected to poor confinement despite placing the sufficient transverse reinforcement is near the outside face of the hollow columns. Furthermore, the limited column displacement ductility capacity was obtained (μ_Δ 2 until 4) with the full lateral pressure induced in the core concrete under the triaxial compression and during cyclic lateral loading in the inelastic range. A significant reduction of confinement and the premature crushing of inside face concrete occurred due to biaxial compression of the circular hollow section [5].

In Tuladhar, Mutsuyoshi, Maki, and Daigo's *[12]* research, One dimensional monotonic and cyclic lateral loading carried out in a single full-scaled spun pile with an outer diameter 300 mm and a wall thickness 60 mm without axial load. Piles embedded in the soil up to 24.8 m from the ground level (GL). The concrete of piles had compressive strength fc' 69 MPa. The effective prestress on the concrete was 5 MPa. Pile's reinforcement used six PC bars of 7 mm diameter for a longitudinal bar and a steel wire with 3 mm of diameter and 100 mm of spacing as transverse steel. This study aimed to determine the plastic hinge locations, which were formed at a depth twice the diameter of the pile (2D) from the ground level during monotonic loading. Meanwhile, due to lateral cyclic loading, the plastic hinge occurred on the pile shaft on the depth 4D under the GL [12].

In Budek and Priestly [3], this study tested four hollow prestressed piles designated with PS11, PS12, PS13, and PS14 had an outer diameter 610 mm with a shell thickness of 94 mm. Samples PS11, PS12, PS13 has an average concrete compressive strength that reached 67.5 MPa. At the same time, PS14 had fc' 53.5 MPa. The effective section prestress was 7.72 MPa. The cyclic lateral load subjected to the midspan of pile combined with constant axial loads 890 kN equals 0,87fc'Ag. The hinge's external confinement was applied to PS11 and PS14 using saddles made from a rubber that covered 100° of pile shaft circumference on the top and bottom surface. The testing result showed that piles failed due to the shell's inability to resist the compression stress when the longitudinal strain at the core's inner surface exceeded more than 0.005. The soil confinement, represented by rubber saddle, and the level of confinement steel ratio on the plastic hinge region do not significantly affect the increase of the displacement ductility capacity of the hollow pile. The bond-slip damage occurred between the surface of non-prestressed longitudinal steel bars and concrete shell. This damage mechanism initiated compression failure and decreased displacement ductility of the hollow pile from μ_Δ 4 to μ_Δ 2.5. Tested piles were recommended to be designed to remain in the elastic condition [3].

A. Codes consideration

The pile foundation seismic design concept is designated to ensure it has adequate strength to resist the seismic force during an intense earthquake. The first stage of designing a pile foundation under seismic load was determining the total deformation system, pile, and soil interaction, during its occurrence using the pushover analysis method. Furthermore, the designing pile had sufficient seismic performance, strength, and ductility, as stated in the previous structural analysis [13].

The seismic performance (SP) of building foundations in Japan consists of three levels: SPI, SPII, and SPIII. According to NEHRP 2000, the pile's seismic performance consists of three levels, with categories A, B, and C corresponding to SPI, while D, E, and F are in SPIII [13, 14]. In the pile foundation seismic design concept, the performance was confirmed by the stability level, which is determined by considering the strength and the deformation of soil and pile members. Tables 1 and 2 show the summary of the ductility factor's limit values for pile according to Japan's code and NEHRP 2000. Both codes illustrated that the minimum ductility factor μ_Δ provided by a pile foundation for high and moderate seismic risk was 8 and 4 / 5.

TABLE I
STATE OF PILE FOUNDATION AND DUCTILITY FACTOR ACCORDING TO
SEISMIC PERFORMANCE LEVELS [13]

Seismic performance	Stability level	State of foundation	Limit values of ductility factor
SPI	Level I	Pile foundation does not yield.	1
SPII	Level II	Although pile foundation yield, they maintain a sufficient bearing capacity.	5
SPIII	Level III	Although the pile foundation reaches the limit state, superstructures do not collapse.	8

TABLE II
LIMIT VALUES OF DUCTILITY FACTOR ACCORDING TO NEHRP 2000 [14]

Seismic risk category	Seismic design category	Limit values of ductility factor
Low	A and B	No requirement
Moderate	C	4
High	D, E, and F	8

II. MATERIALS AND METHOD

A. Specimen Dimensions

Six spun piles with concrete infill cast inside the spun pile were tested with different axial loads, namely 392 kN (0.08fc'Ag) for S-DB-1 S-DB-2, S-DB-5 and 784 kN (0.16fc'Ag) for S-DB-3, S-DB-4, S-DB-6. The dimensions of the specimens are shown in Fig. 3. The spun pile's outer

and inner diameters at 400 mm and 200 mm produce a thickness (t/D = 0.25) value of 100 mm of the thickness, as shown in Fig. 3. One layer reinforcement consisted of ten bars of PC bar ∅7.1 mm were embedded to pile and placed near the outside face of the section with a concrete cover of 35 mm. The transverse steel bars used steel wire with diameter ∅3.2 mm and 100 mm of spacing. The pile's cross-section and reinforcement's detail are shown in Fig. 3.
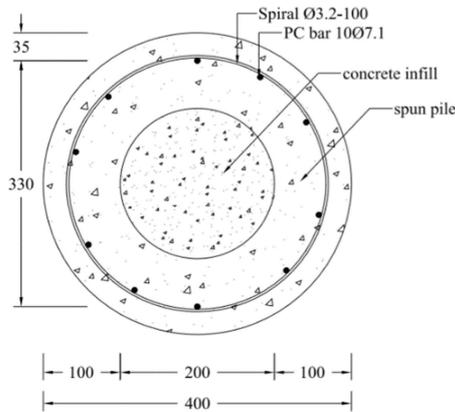

Fig. 3 Cross-section and reinforcement detail of specimens

B. Properties of materials

The specimens consist of the spun pile and infill concretes. The compressive strength at 28-days age of concrete of spun pile and concrete infill was 54.4 MPa and 33.0 Mpa, respectively. Fig. 4 shows the PC's tensile properties with a bar and spatial wire diameters of 7.1 mm and 3.2 mm. The PC bar's cross-sectional area and the wires were 39.6 mm2 and 8 mm2, with a modulus of elasticity of 229,577 Mpa, and 219,774 Mpa. The PC bar reached yield, and the ultimate condition at stress level was 1,387 Mpa and 1,455 Mpa, while wire attained ultimate state at a stress level of 712 Mpa. The PC bar's yield and ultimate strain were 0.007 and 0.023, with an ultimate strain of 0.02.
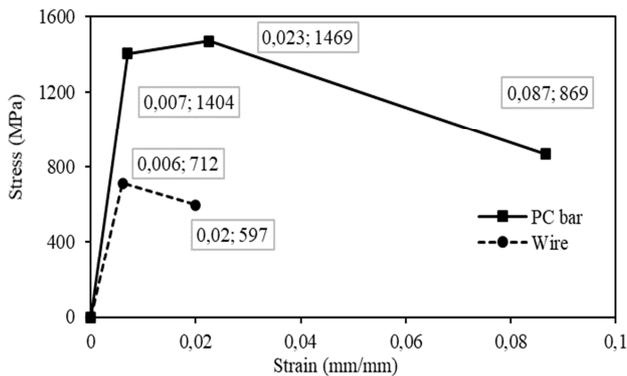

Fig. 4 Tensile stress-strain relationship of PC bar and spiral reinforcement

C. Loading Setup

The spun pile, which is vertically simply supported, was tested, as shown in Fig. 5a. The picture of the testing setup in the laboratory was depicted in Fig. 5b. Both ends of the pile were supported using a pin. For transferring the axial force generated by hydraulic compression stressing jack to the pile's shaft, upper hole support was designed as an oval

shape. Constant axial loads, 392 kN (0.08 fc'Ag), and 784 kN (0.16 fc'Ag) represent gravity loads subjected to the top of the spun pile. The reverse lateral load, representing the seismic load, was imposed as two points loading in the spun pile's middle span using a push and pull actuator.

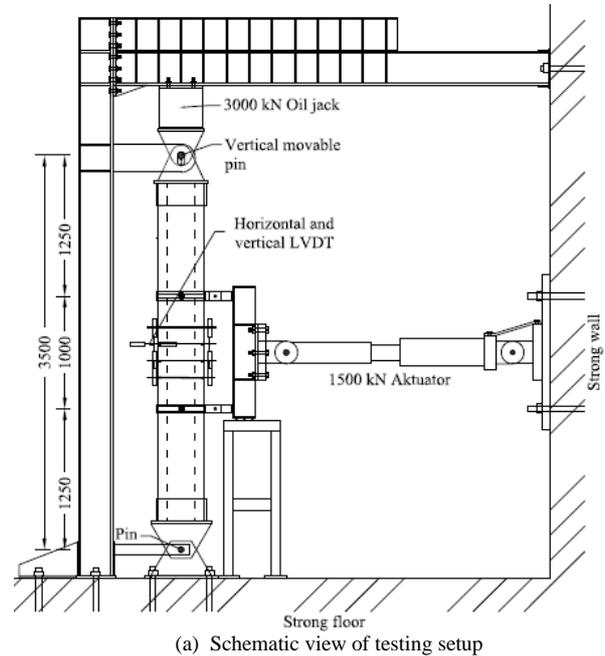(a) Schematic view of testing setup

(b) Implementation in laboratory

Fig. 5 Test setup of specimen and loading system for cyclic flexural with axial load testing

The lateral displacement was measured using a 100 mm Linear Variable Differential Transducer (LVDT) mounted in the midspan. The value of the drift ratio obtained by dividing the recorded lateral displacement and half of the shear span. Furthermore, sequence cycles of the drift ratio for reverse cyclic loading of ACI 374-1-05 were shown in Fig. 6. Constant reverse displacement was repeated in three cycles, with an increase in drift ratio until the pile failed [15].
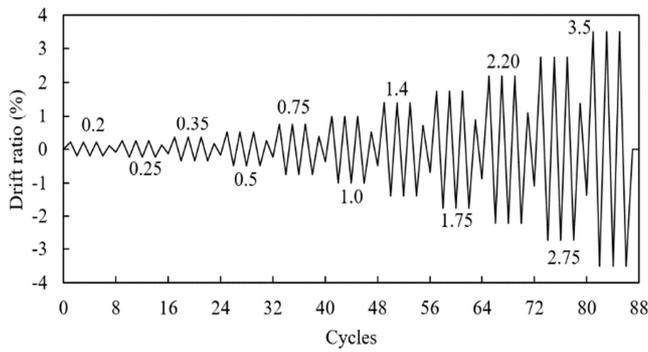
Fig. 6 The Sequence of drift ratio according to ACI 347-1-05 [15]

III. RESULTS AND DISCUSSION

A. The Curve of Hysteresis Loops of The Load-Displacement

The hysteresis curves of the load-displacement relationship of tested spun pile under combined reversed flexural and constant axial load were shown in Fig. 7 and 8. Two cases were analyzed from these curves, with the first regarding the displacement ductility factor. In contrast, the second is related to the seismic performance of spun pile according to Japan's code and NEHRP 2000.

The spun pile's average displacement ductility factor with concrete infill tested under axial loads of 392 kN (0.08f'cAg) and 784 kN (0.16f'cAg) was 5.8 and 3.7, respectively as shown in Table 3. The ordinary spun pile i.e. without concrete infill had an average displacement ductility 4.9 and 2.6, under axial loads 0.08f'cAg and 0.16f'cAg, respectively [16]. Therefore, due to the concrete infill's presence, the displacement ductility increased by 18% when loaded with 0.08f'cAg and 42% at 0.16f'cAg of axial loads.

TABLE III
THE SUMMARY OF DISPLACEMENT DUCTILITY FACTOR OF SPUN PILE WITH CONCRETE INFILL

Axial load	Sample	Displacement ductility, μ_Δ	μ_Δ average
392 kN (0.08fc'Ag)	S-DB-1	5.9	5.8
	S-DB-2	5.8	
	S-DB-5	5.7	
784 kN (0.16fc'Ag)	S-DB-3	3.1	3.7
	S-DB-4	3.9	
	S-DB-6	4.1	

TABLE IV
THE SUMMARY OF DISPLACEMENT DUCTILITY FACTOR OF SPUN PILE WITHOUT CONCRETE INFILL [16]

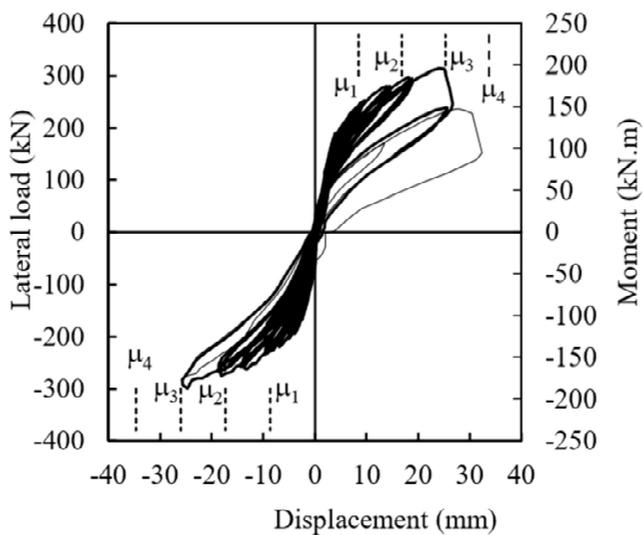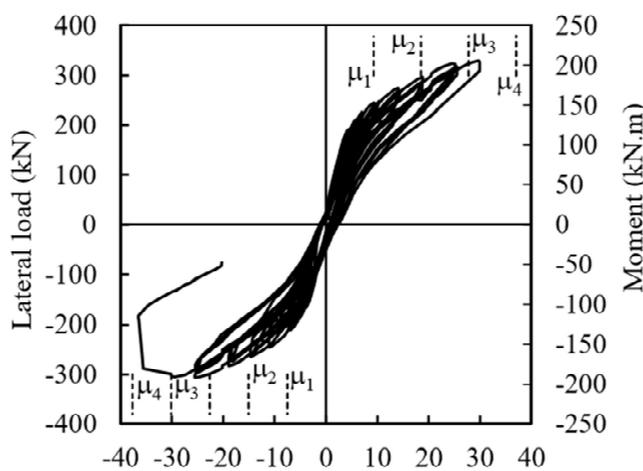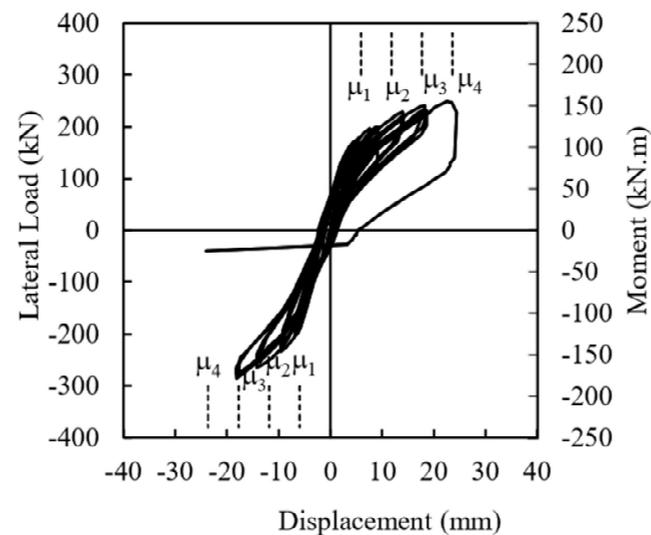| Axial load	Sample	Displacement ductility, μ_Δ	μ_Δ average
392 kN (0.08fc'Ag)	S-TB-1	4.8	4.9
	S-TB-2	4.9	
784 kN (0.16fc'Ag)	S-TB-3	2.4	2.6
	S-TB-4	2.8	
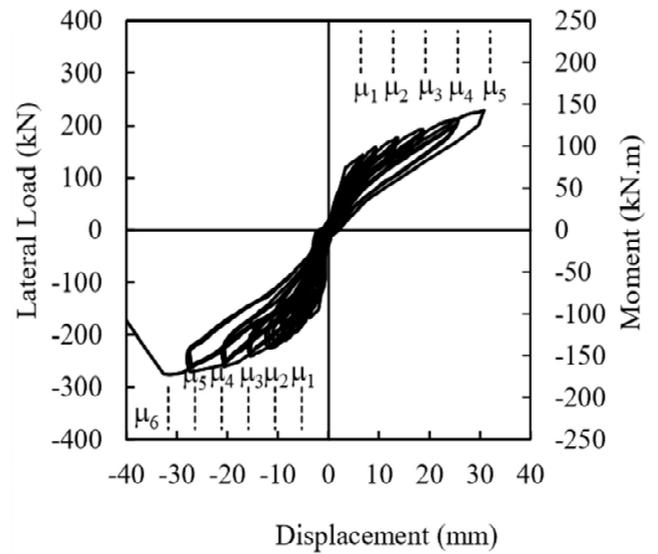
a) S-DB-1
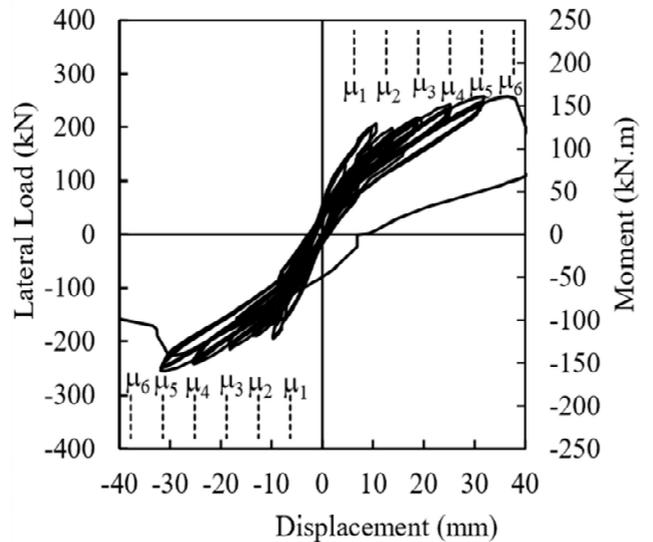
b) S-DB-2
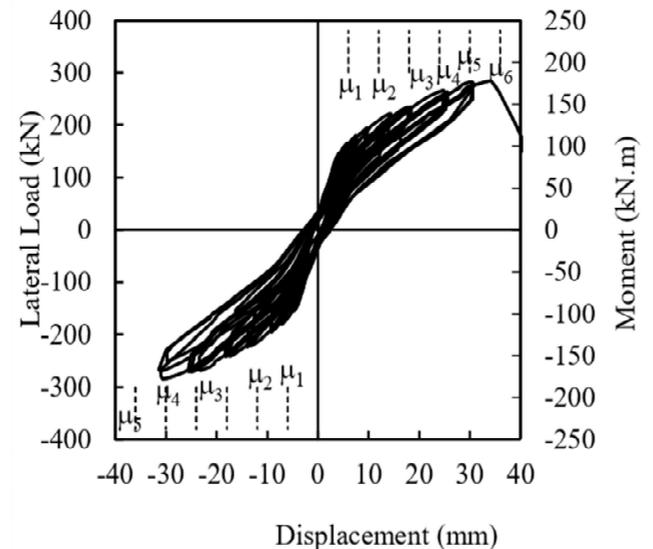
c) S-DB-5

Fig. 7 The hysteresis load-displacement of the spun pile with concrete infill under 392 kN (0.08fc'Ag) of axial load

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a) S-DB-3

b) S-DB-4

c) S-DB-6

Fig. 8 The hysteresis load-displacement of the spun pile with concrete infill under 784 kN (0.16fc'Ag) of axial load

B. Displacement Analysis of The Ductility Factor According to Seismic Code Requirements

The ductility factor's displacement analysis as the spun pile's seismic performance parameter was carried out by comparing the ductility factor with the limit factor requirements regulated by seismic design codes. The purpose of this analysis was to classify the appropriate application of tested pile according to the seismic risk category (SRC) or Seismic Design Category (SDC) and its stability level.

According to the previous analysis, the spun pile's average displacement ductility factor with concrete infill was 5.8 for specimens tested under 0.08fc'Ag axial load and 3.7 for 0.16fc'Ag, as shown in Fig. 9. The piles of foundations were permitted to attain yield state with sufficient bearing capacity by using a stability level of II. Meanwhile, applying 0.16fc'Ag of an axial load on tested spun pile, obtained a ductility level of less than 5, due to the stability of the caused pile foundations in level I. These piles were used in low seismic risk category which means that 0.16fc'Ag axial load-induced spun pile needs to be designed in an elastic state. At 0.08fc'Ag, it was permitted to be designed as partial ductile members.
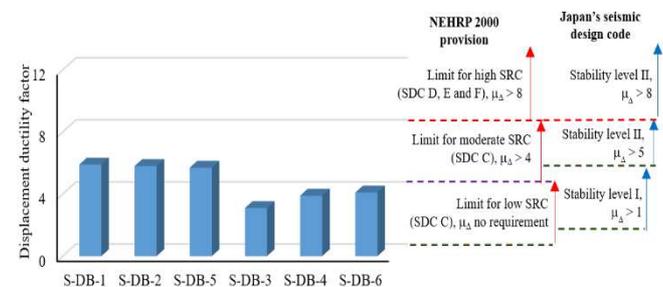

Fig. 9: State of displacement ductility factor of tested specimens according to seismic codes requirement

IV. CONCLUSION

This experimental research provides several illustrations showing the influence of concrete infill on the spun pile's flexural performance. Seismic performance of spun pile with concrete infill can be used up to medium risk when the axial loads were 392 kN (0.08f'cAg) and 784 kN (0.16f'cAg) with displacement ductility of 5.8 and 3.7, respectively. The presence of concrete infill increases the average spun pile displacement ductility by 18% when loaded with 392 kN (0.08f'cAg) and 42% at 784 kN (0.16f'cAg) axial loads. According to Japan's code and NEHRP 2000 requirement, tested spun piles with concrete infill were appropriately used in moderate and low seismic risk categories under 0.08fc'Ag and 0.16fc'Ag axial loads. Further research is needed to investigate the effect of providing adequate spiral reinforcement to confine concrete core. The objective is to obtain a displacement ductility factor more than 8. Therefore, the spun pile has sufficient ductility to be applied to the high seismic risk category.

NOMENCLATURE

f	stess	MPa
A	area	mm^2

Greek letters

μ   ductility factor
ρ   ratio of reinforcement
∅   diameter of steel bars

Subscripts

Δ   displacement
c   concrete
g   gross

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