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통권 제37호 2015년 12월 발행,
1988년 7월 25일 등록, 번호(서)바-98
ISSN 1229-8034

대우건설기술

Daewoo Construction Technology Report



드론을 활용한 3D 스캐닝

카메라와 각종 센서를 장착한 드론을 통해서
완벽한 현장 모니터링이 가능해지고 있다.
특히, 현장 측량이 자동화 되며, 굴토량 등
현장시공물량이 간편하게 계산될 수 있고,
3차원 입체 모델링을 이용한
3D프린팅을 이용해 효과적인 공정관리 및
시공계획 수립이 가능해질 것이다.

행복은 가까이에 있다
풀밭 위에도
공기 중에도
저 하늘에도

날씨 좋은 날이면
친구들과 잔디 밭에 누워
아무 생각 없이 바람을 쐬고
잠시 그렇게 여유를 즐기는 것,
일상 속 행복이란 이런 것 아닐까요?

마음껏 누워 쉴 수 있는 잔디밭
깊이 마실 수 있는 맑은 공기
그래서 당신이 더 행복해 지는 세상
대우건설이 함께 만들겠습니다

세상을 바꾸는 힘
대우건설



통권 제37호

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대우건설기술연구원

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토목

- Lateral Behavior of Monopile for Offshore Wind Power Using Centrifugal Model Tests on Sandy Soil
- Experimental and Numerical Study of Wind Loads for Newly-proposed Steel Wind Turbine Towers
- 흙도로포장용 재료로서 폐콘크리트 미분말의 활용성 연구
- 요철형 이음단면을 갖는 프리캐스트 교량 바닥판의 휨성능평가



Lateral Behavior of Monopile for Offshore Wind Power using Centrifugal Model Tests on Sandy Soil

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본 논문은 2015년 ISOPE(국제해양및극지공학회) 학술발표회, 2015-06-22, pp. 212-217에 게재되었던 논문을 재게재한 것입니다.

Abstract

In this study, lateral static and cyclic loading tests were performed using the centrifugal model tests in sandy soil to evaluate lateral behavior of monopile for offshore wind power. The model monopile was made from aluminum and measured 624mm in length, 35mm in diameter. During the centrifugal model tests, the gravity acceleration of 94.2g was applied and so the monopile was described to have the actual length of 60m and diameter of 3.3m. Tests were performed on dry Joomunjin standard sand with the relative density of 40% and 80% to obtain lateral bearing capacity and deformation of monopile with depth. Based on the test results, prototype static and cyclic p-y curve was proposed by considering the API RP 2A-WSD(2010).

Keywords : offshore wind power , monopile, lateral static and cyclic loading, p-y curve.

1_INTRODUCTION

A lot of studies on an analysis of lateral bearing behavior of pile utilizing scale model test have been made in Korea and foreign countries until now. However, a general scale model test is conducted, only satisfying the similitude relationship of prototype shape in 1g gravitational field condition. Accordingly, it is shown that actual stress-strain behavior due to the dead weight of soil is different from that of in-situ ground. Besides, most of test results are about pile of 1m or below in diameter, which can be applied to onshore. So, there is a limitation in applying those to predicting the bearing behavior of large monopile which is mainly used as an offshore wind turbine support structure.

Hence, in this study, a static and cyclic loading test was conducted in lateral direction, changing the relative density of soil to 40% and 80% respectively, for a pile of 3.3m in diameter and 60m in length when converted on the basis of circular shape by using a centrifuge in order to overcome the above limitation. Through the process, the ultimate lateral bearing capacity of monopile was evaluated. And a p-y curve according to depth was derived, and then was compared with API RP 2A-WSD (2010) p-y curve.

2_CENTRIFUGAL MODEL TEST

With regard to centrifugal model test, the centrifuge of DAEWOO E&C was utilized. Centrifuge is 3m in radius, and can accelerate a mass of 1.2ton up to 100g of centrifugal acceleration. Fig. 1 is a

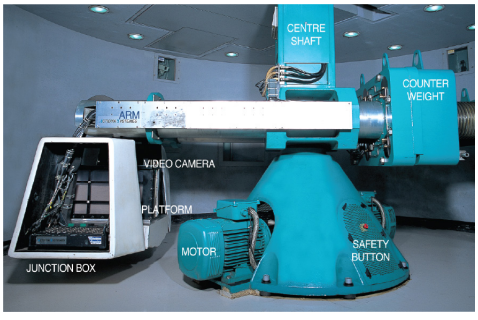


Fig. 1 Centrifuge (DAEWOO E&C)

view of the entire centrifugal test equipment used for the test.

2.1. Static loading test equipment

Fig. 2 shows a soil box used for static loading test. This soil box is 80(W) × 50(H) × 20(B)cm. And the bottom and three sides were made of stainless steel plate. And the front is composed of acrylic plate of 4cm in thickness. It was so manufactured that the condition of soil can be observed through that.

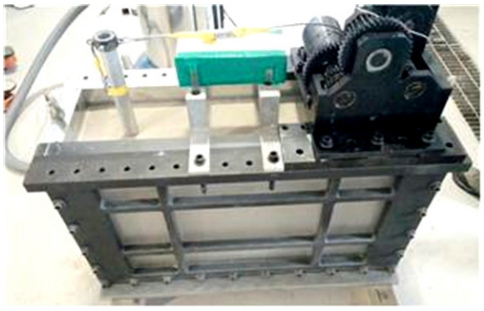


Fig. 2 Static loading test soil box

2.2. Static loading test procedure

When conventional static loading test was considered, static loading tests of pushing a monopile by using an actuator were conducted in large numbers. However, this study devised a method of pulling the upper part of pile as a wire is wound by using an electric motor after the electric motor is connected to the pile by using the wire. Fig. 3 shows a schematic diagram of test procedure. In order to measure deflection and load in the upper part of pile, load cell and LVDT was installed. Besides, in order to analyze the lateral behavior of monopile, strain gauges were attached to each place in two directions, and then strain was measured regarding a depth in a total of 8 places.

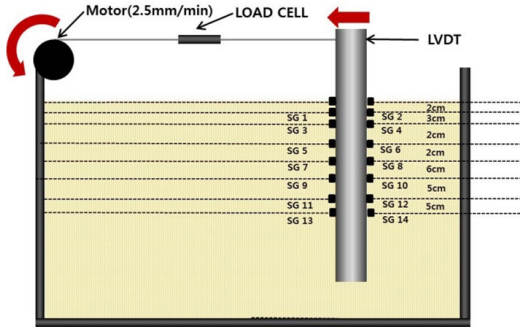


Fig. 3 Schematic diagram of static loading test

A static loading test was conducted as shown in the following Table 1. The static loading test of pile embedded at a depth of 400mm was conducted at a motor speed of 2.5mm/min at relative soil density of 40% and 80%.

Table 1. Static loading test conditions

Classification	Penetration depth (mm)	Motor speed (mm/min)	Gravity (g)	Relative density (%)
Case 1	400	1.06	1.15	40
Case 2				80

2.3. Cyclic loading test equipment

Fig. 4 shows a soil box used for cyclic loading test. The soil box used for cyclic loading test is 80(W) × 60(H) × 50(B)cm. Four sides of strong box were made of high-strength aluminum alloy, and were so manufactured that those could be used for centrifugal model test.

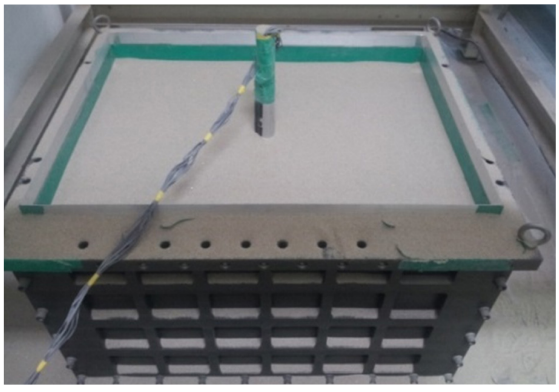


Fig. 4 Cyclic loading test soil box

2.4. Cyclic loading test procedure

When a cyclic loading test is considered, cyclic loading test equipment using a cam was devised as shown in Fig. 5(b), on the basis of conventional 1-g cyclic loading test equipment devised by Peng et al. (2006) as shown in Fig. 5(a). As the significant features of devised equipment were considered, the equipment was so designed that it could withstand gravity acceleration up to 100g, and

lateral load magnitude was enabled to be adjusted by changing weights on both sides. Besides, the extension distances of pile can be adjusted, and cam system is used. Accordingly, the period of cycle can be adjusted by adjusting the rotational speed of cam.

Fig. 6(a) is a schematic diagram of cyclic loading test. And Fig. 6(b) is a view of the entire cyclic loading system. As the cam rotates, the load of weight hanging on the side of soil box is transmitted to the pile, and then a horizontal cyclic load is applied. Regarding this, in order to analyze the lateral cyclic behavior of pile, load cells and LVDTs were installed in 2 places respectively. And strain gauges were installed in 8 places respectively. With regard to the number of strain gauges and place where those were attached, strain gauges were attached in the same way as that of static loading test.

A cyclic loading test was conducted as shown in Table 2. The frequency of cam was set for 0.125Hz. And a total of 3 types of test load were set. This load was estimated to be 30, 50, and 80% of ultimate load obtained from static loading test. Table 2 summarized the test conditions of cyclic loading test.

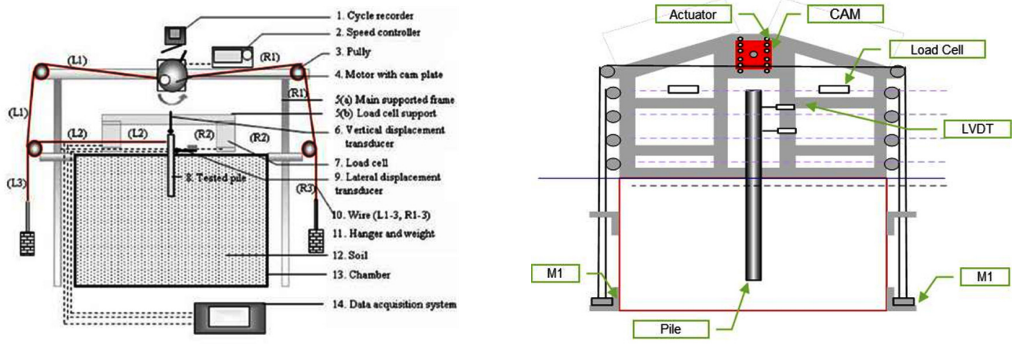
3_MATERIAL

3.1. Soil

Joomunjin standard cohensionless sand was used in the experiments presented as part of this study. It is classified as a poorly

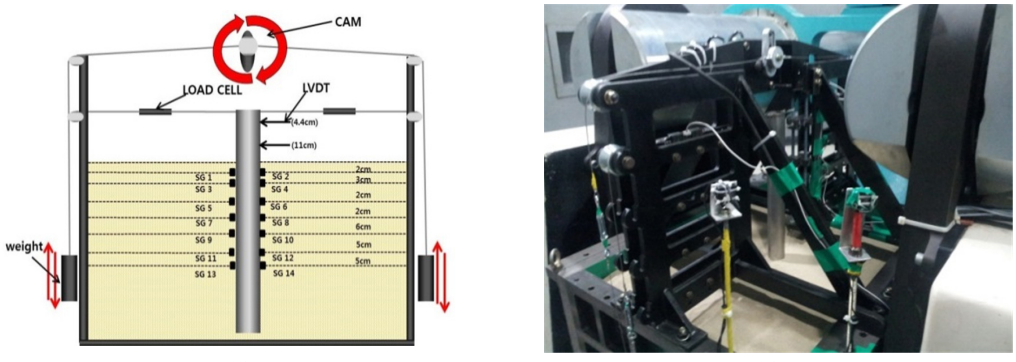
Table 2. Cyclic loading test conditions

Classification	Penetration depth (mm)	Weight (g)	Gravity (g)	Relative density (%)
Case 3	400	130	94.2	40
Case 4		187		
Case 5		340		



(a) (b)

Fig. 5 (a) System devised by Peng et al. (2006) (b) System devised by DAEWOO E&C



(a) (b)

Fig. 6 (a) A schematic diagram of cyclic loading test (b) A view of the entire cyclic loading system

graded sand (SP) according to Unified Soil Classification System (USCS) and can be characterized used the following parameters: Specific gravity (Gs) is 2.65; maximum dry unit weight measured 16.6 kN/m^3 and minimum dry unit weight measured 13.3 kN/m^3; and the internal friction angle measured 30 and 38 degrees at a relative density of 40% and 80% respectively. Table 3 shows the material properties of Joomunjin standards and used for the test.

Table 3. Properties of soil

USCS	SP	
Maximum unit weight (kN/m^3)	16.6	
Minium unit weight (kN/m^3)	13.3	
Coefficient of uniformity (Cu)	1.68	
Specific gravity (Gs)	2.65	
Relative density (%)	40	80
Friction angle	30	38

3.2. Pile

Fig. 7 shows a monopile used for the test. With regard to the materials of pile, aluminum of 70GP in elastic modulus was used. It was 624mm in length, and 35mm in outer diameter, and 29mm in inner diameter. The test was conducted at gravity acceleration of 94.2g. Therefore, when the dimensions of model monopile were converted into those of prototype, a steel monopile of 3.3m in diameter and 60m in length was simulated.



Fig. 7 Monopile used for test

4_TEST RESULT

4.1. Ultimate lateral capacity

The test values of load-deflection relation obtained through the static loading test were converted on the basis of gravity acceleration of 94.2g, and were analyzed as shown in Fig. 8. As suggested by Fleming (1992), a value in case deflection in pile cap occurred by 10% of diameter was estimated to be ultimate lateral bearing capacity as shown in Fig. 8. As a result, we calculated that ultimate lateral bearing capacity was 3,474kN and 4,722kN at 40% and 80% respectively.

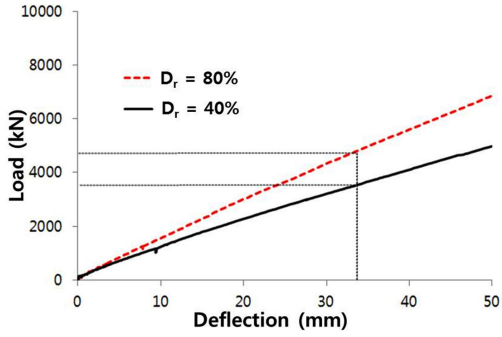


Fig. 8 The calculation of ultimate lateral bearing capacity according to relative density

4.2. Static p-y curve

In the test, strain gauges were attached according to each depth of pile, and then the behavior of pile was measured. Through the process, a static p-y curve was estimated by using the equation (1), the equation of hyperbola presented by Kondner (1963). There are 2 variables for equation (1). And those are initial soil resistance coefficient (kini) and ultimate soil resistance (pu). Those can be obtained through equation (2) and (3) presented by Broms(1964).

$$p = \frac{y}{\frac{1}{k_{ini}} + \frac{y}{P_u}} \quad (1)$$

$$k_{ini} = n_h z^n \quad (2)$$

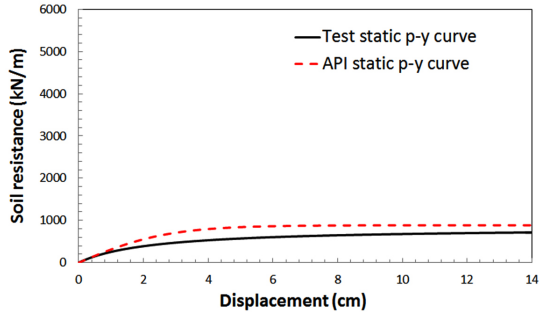
$$p_u = A_{\gamma'} k_p z^n D \quad (3)$$

Where, A, n: Coefficient, Y' : Unit weight of soil, K_p : Passive earth pressure coefficient, z : Depth

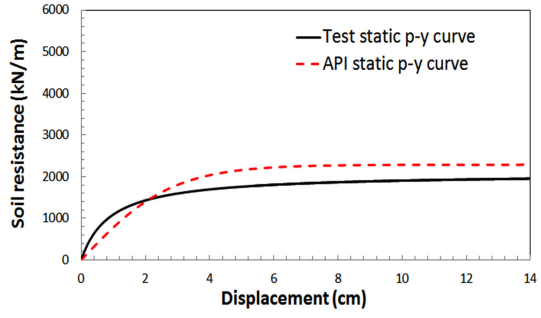
Fig. 9~10 is a figure that compares API(2010) static p-y curve with static p-y curve calculated through test in sandy soil where relative density is 40% and 80% respectively.

As shown in Fig. 9, the initial slope of p-y curve of centrifugal model test was estimated to be slightly steeper than, or similar to that of API curve. And ultimate soil resistance of p-y curve of centrifugal model test and that of API curve were similarly converged. . Ultimate soil resistance at relative density of 40% showed a result similar to that of API static p-y curve.

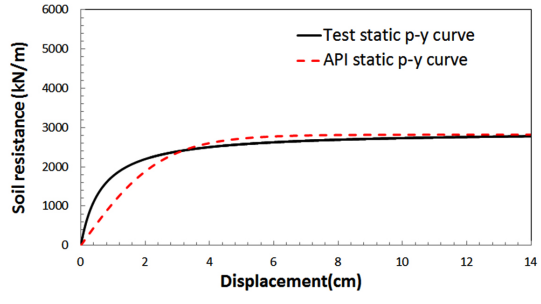
As shown in Fig. 10, the initial slope of p-y curve of centrifugal model test was estimated to be similar that of API curve. However, an analysis showed that a differences in ultimate soil resistance increased as depth increased at relative density of 80%. And ultimate soil resistance of test p-y curve was estimated to be smaller than that of API static p-y curve. This is because soil was destroyed early during the course of the experiment. So, it is analyzed that the ultimate soil resistance was not occurred sufficiently. As shown in



(a)

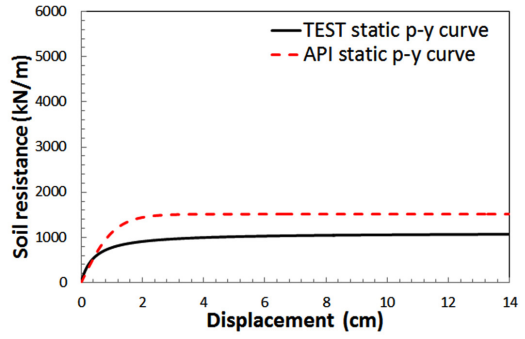


(b)

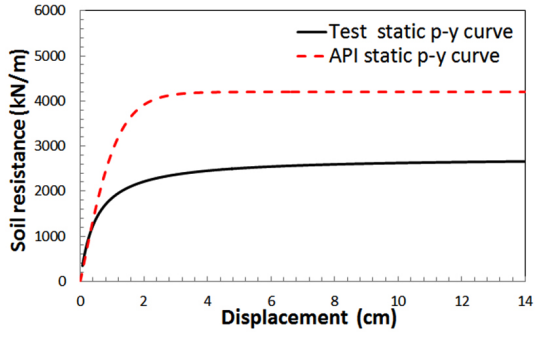


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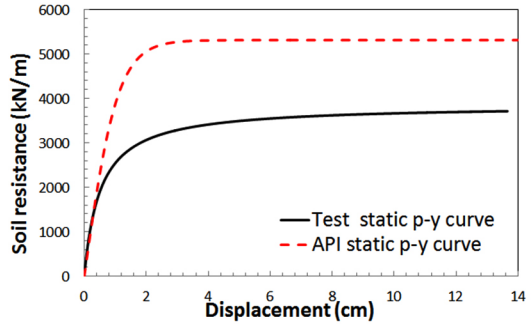
Fig. 9 p–y curve at each depth in soil having D_r=40% (a) Depth=2m, (b)Depth=5m, (c)Depth=7m



(a)



(b)



(c)

Fig. 10 p–y curve at each depth in soil having Dr=80%(a) Depth=2m,(b)Depth=5m,(c)Depth=7m

Fig. 9~10, the initial slope of soil resistance and ultimate soil resistance increased as relative density increased and as depth increased according to the previous tendencies in the analysis.

4.3. Cyclic p-y curve

In this test, in order to estimate a cyclic p-y curve, the deformation of pile was measured by using a strain gauge, and was substituted for variable in equation (4), and then bending moment was obtained. A moment curve was estimated by using cubic spline interpolation method (Dou & Byrne, 1996), and then p-y curve at each depth was estimated by using beam theory equations as shown in equation (5)~(6).

$$M = \frac{E\varepsilon I}{y} \tag{4}$$

$$p = \frac{d^2}{dz^2} M(z) \tag{5}$$

$$y_{pile} = \iint \frac{M(Z)}{EI} dz \tag{6}$$

Where, p is soil resistance, and y *pile* is the displacement of pile, and y is the radius of pile.

Fig. 11 shows a cyclic p-y curve at pile penetration depth of 2m according to the cyclic loading magnitude at the same frequency. With regard to cyclic loading, we used 30, 50, and 80% of 3,474kN, a value of ultimate lateral bearing capacity at relative density of 40%, which was obtained in the previous static loading. When (a) ~ (c) in Fig. 11 is compared, it was shown that displacement and soil resistance magnitude tended to increase as cyclic loading magnitude increased in the analysis. Here, the frequency of cyclic loading is equal to each other. Accordingly, each maximum soil resistance points can be expressed on the single cyclic p-y curve. And this can be displayed as shown in Fig. 12.

Fig. 12(a) ~ 12(b) compares API cyclic p-y curve with maximum soil resistance points at the depth of 2m and 7m respectively. It was shown that the slope and ultimate soil resistance of centrifuge results was larger than that of API cyclic p-y curve at two depths. And when depth was 2m, nonlinear displacement occurred within 1% of pile diameter. And when depth was 7m, the nonlinear displacement of pile behavior didn't occur. The slope

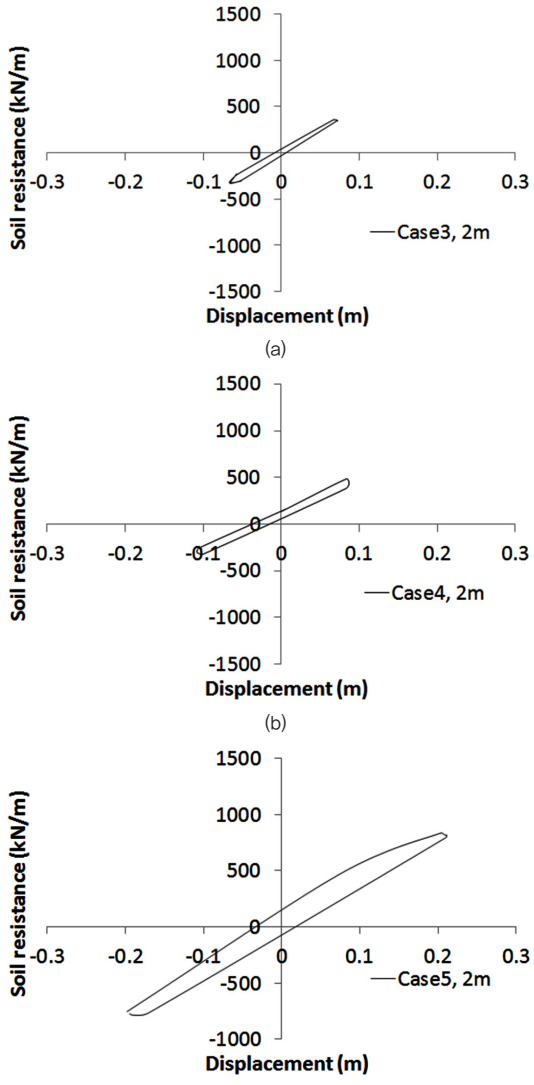
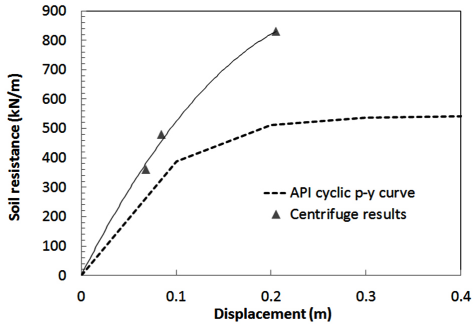
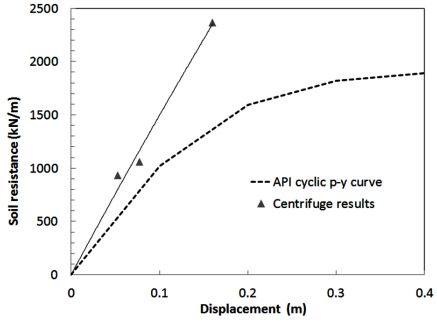


Fig. 11 Cyclic p–y curve at depth of 2m in soil having Dr=40 (a) Lateral Load=1,224kN (Model Load: 130g), (b) Lateral Load=1,761kN(Model Load: 187g), (c) Lateral Load=3,202kN(–Model Load: 340g)

and ultimate soil resistance of p-y curve by centrifuge tests being steeper and larger than that of API cyclic p-y curve. In calculation of API cyclic p-y curve, this is because factor A to account for cyclic loading condition apply all depth by 0.9, which recommended API(2010).



(a)



(b)

Fig. 12 Centrifuge results vs API cyclic p–y curve (a) Depth = 2m, (b) Depth = 7m

5_CONCLUSIONS

In this study, a static and cyclic loading test was conducted in lateral direction according to the relative density of sandy soil in the condition of gravity acceleration of 94.2g by using a centrifugal model test in order to evaluate the lateral bearing behavior of monopile, and ultimate lateral bearing capacity and p-y curve was examined.

1) A value in case deflection in pile cap occurred by 10% of diameter was estimated to be ultimate lateral bearing capacity according to the theory presented by Fleming (1992). As a result, we calculated that ultimate lateral bearing capacity was 4,722kN and 3,474kN at relative density of 80% and 40% respectively.

2) In case of being compared with API (2010) p-y curve, in sandy soil at relative density of 80%, an analysis showed that an error in ultimate soil resistance increased as depth increased.
Ultimate soil resistance at relative density of 40% showed a similar result up to the depth of 7m in the analysis. It was shown that the initial slope of soil resistance increased as rela tive density increased and as depth increased in the analysis.

3) A maximum soil resistance point was obtained at a depth of 2m and 7m according to horizontal cyclic load magnitude after deciding that 30, 50, and 80% of ultimate lateral bearing capacity at relative density of 40% was horizontal cyclic load, and then it was compared with that of API (2010) cyclic p-y curve.

4) It was shown that the initial slope and ultimate soil resistance of test results tended to be larger than that of API cyclic p-y curve. And nonlinear displacement occurred at a depth of 2m within the displacement of 1% of pile diameter. And linear displacement occurred at a depth of 7m.

ACKNOWLEDGEMENT

This research was a part of the project titled "Development of Substructure Systems for Offshore Wind Power in Shallow Sea Water (Less than 40m)/2011T100201105" funded by the Ministry of Trade, Industry and Energy, Republic of Korea.

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Experimental and Numerical Study of Wind Loads for Newly-proposed Steel Wind Turbine Towers

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본 논문은 2015년 Journal of Ocean and Wind Energy Vol.2, No.3, pp. 153–159에 게재되었던 논문을 재게재한 것입니다.

Abstract

In this study, a wind tunnel test and a Computational Fluid Dynamics (CFD) simulation were carried out for a steel modular tower and a multi-column tower, which were proposed by the Research Institute of Industrial Science & Technology (RIST) in Korea, in order to obtain their mean wind force coefficients and to verify the reliability of the obtained mean wind force coefficients through a comparison of the results from the wind tunnel test and CFD simulation, which were carried out by the Daewoo Institute of Construction Technology. The proposed steel modular tower has an octagonal cross section for easy transportation in the phase of construction. The proposed steel multi-column tower consists of four columns mounted on top of a conventional circular-type main column, with a smaller cross-sectional area relative to the main column to mitigate the wind load on the tower. Through the wind tunnel test and CFD simulation, the towers' mean wind force and moment coefficients, which represent the resistance of an object in the air, were obtained and compared with one another to verify the reliability of the obtained mean wind force coefficients.

Keywords : steel modular tower, multi-column tower, wind tunnel test, CFD, drag coefficients.

1_INTRODUCTION

Recently, the size of blades is getting larger than those of the past due to the high demand for large-capacity wind turbines. The increasing size of blades to meet the high demand for large-capacity wind turbines leads to the increasing size of the supporting structures. Due to their increasing size, the blades are inevitably exposed to larger amounts of loads (e.g., thrust loads, wind loads, wave loads, and so on), and therefore the supporting structures, including the towers and foundations, are required to be designed more efficiently from the cost perspective. The investigation of wind loads on wind turbine towers is one of the necessary steps in designing them.

Previous studies of wind loads on wind turbine towers were mainly done with a conventional circular cross-sectional shape tower. A method was proposed to estimate the design wind loads on the blades and tower through a wind tunnel test (Kawai et al., 2008). The flow field behind the wind turbine towers was investigated numerically (Hagen et al., 2011). A wind tunnel study was conducted to quantify the wind loads and evolution of the unsteady vortex structures in the near wake of a wind turbine model (Hu et al., 2012).

In this study, a wind tunnel test and a Computational Fluid Dynamics (CFD) simulation were carried out for a steel modular tower and a multi-column tower, which were proposed by the Research Institute of Industrial Science & Technology (RIST) in

Korea, in order to obtain their mean wind force coefficients and to verify the reliability of the obtained mean wind force coefficients through a comparison of the results from the wind tunnel test and CFD simulation. The feasibility study of the proposed towers for a 10 MW wind turbine is currently being researched with funding from the Korean Government. The proposed steel modular tower has an octagonal cross section for easy transportation in the phase of construction. The proposed steel multi-column tower consists of four columns mounted on top of a conventional circular-type main column, with a smaller cross-sectional area relative to the main column to mitigate the wind load on the tower. The results were compared with one another to verify the reliability of the obtained mean wind force coefficients, which represent the resistance of an object in the air or water and depend on the Reynolds number.

2_WIND TUNNEL TEST

The wind tunnel test was carried out at the Daewoo Institute of Construction Technology. The wind tunnel laboratory is a boundary layer circuit-type wind tunnel with test area dimensions of 3 m × 2 m × 20 m (width × height × length) and a wind speed capacity of 0.5 m/s~30 m/s. The schematic view of the wind tunnel test laboratory is shown in Fig. 1.

Two different methods of the wind tunnel test were conducted in this study. For the wind tunnel tower models of a conventional circular tower, a modular tower with an octagonal cross section, and a multi-column tower consisting of four columns with a smaller cross-sectional area relative to the main column, a 1/300 scale was chosen to reduce the wind tunnel blockage effects that can be caused by rotating blades in the wind tunnel test in the next section. The wind tunnel model was attached to a six-component load cell, which was located at the bottom of each model, to measure the forces and moments under a uniform wind flow. Especially for the multi-column tower, additional wind tunnel section models were made to investigate the effect of the distance between the four columns on the value of the wind force coefficients. A 1/100 scale was chosen to ensure that the model was stiff enough not to vibrate during the wind tunnel test. To measure the forces on the wind tunnel model, a three-component load cell was attached to the end of the model. A detailed description of the wind tunnel tests described above is given in the next section.

2.1. Outline of Wind Tunnel Experiment

The 1/300-scale wind tunnel models were exposed to a uniform wind flow in the wind tunnel test. The wind speed is set to 10 m/s, which is the same value as that for the CFD simulation. The real-scale models are shown in Fig. 2 for a conventional circular tower, a modular tower with an octagonal cross section, and a multi-column tower consisting of four columns with a smaller cross-sectional area relative to the main column. The height of the models is 120 m, and the bottom width of the cross section is 7.75 m. A six-component load cell of a strain gauge type was used to measure the forces and moments for a small-scale model in the wind tunnel test. It was highly convenient for the measuring forces and moments to be located at the bottom of the small-scale

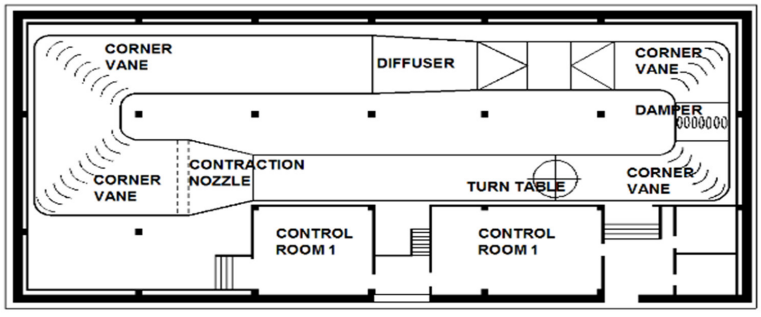
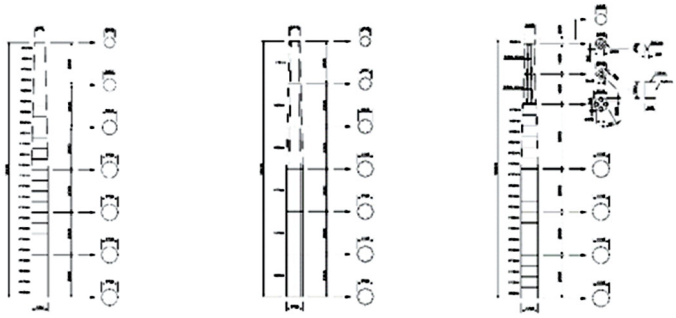


Fig. 1 Schematic view of the wind tunnel test laboratory.



(a) a conventional circular tower (b) a modular tower (c) a multi–column tower

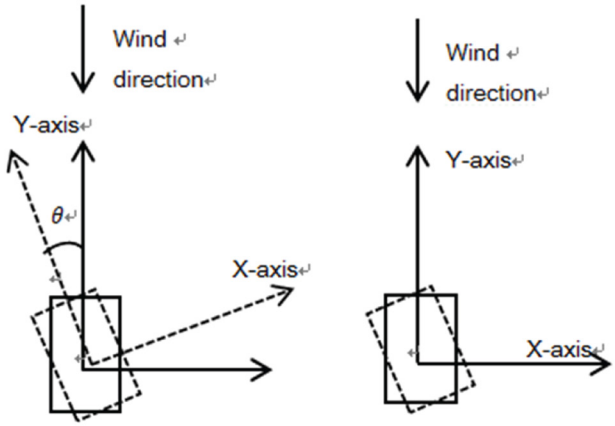
Fig. 2 A conventional circular tower, a modular tower, and a multi–column tower.



Fig. 3 Model setup

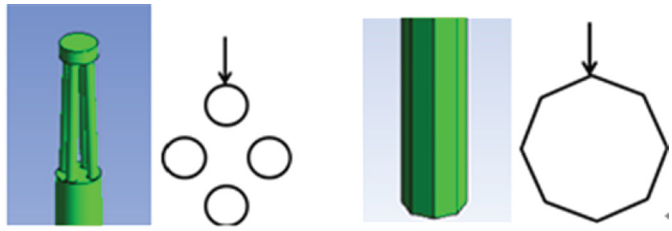
model. The model setup is shown in Fig. 3.

In this study, two different definitions of axes are used here: structural axes and wind axes. Structural axes are defined in Fig. 4a and changed according to the rotation angle θ. However, wind axes remain in their original position at 0 degrees and are unchanged according to θ as shown in Fig. 4b. The turntable to which the experimental model is attached is set to be rotated stepwise by 10 degrees in an anticlockwise direction, as shown in Figs.4a and 4b, and the forces and moments are measured at each 10 degrees until 350 degrees is reached. The initial position of the models at 0 degrees is shown in Fig. 5. The forces and moments for the structural axes are directly measured from the six-component load cell attached to the small-scale model. The mean wind force and moment coefficients for the structural axes and wind



(a) Structural axes (b) Wind axes

Fig. 4 Definition of structural axes and wind axes.



(a) a steel modular tower (b) a multi-column steel tower

Fig. 5 Initial position of models at 0 degrees

axes are calculated by Eqs. 1 to 4, respectively:

$$C_{F,x} = \{\overline{F_x}/q_H BH\} = \text{Force coefficient at x-axis} \quad (1)$$

$$C_{F,y} = \{\overline{F_y}/q_H BH\} = \text{Force coefficient at y-axis} \quad (2)$$

$$C_{Mx} = \{\overline{M_x}/q_H BH^2\} = \text{Moment coefficient at x-axis} \quad (3)$$

$$C_{My} = \{\overline{M_y}/q_H BH^2\} = \text{Moment coefficient at y-axis} \quad (4)$$

where

$\overline{F_x}$, $\overline{F_y}$ = Mean forces at x-axis and y-axis

$\overline{M_x}$, $\overline{M_y}$ = Mean moments at x-axis and y-axis

B = reference width of a structure

H = reference width of a structure

$$q_H = \frac{1}{2}\rho V^2$$

where

ρ = air density

V = mean wind speed

For the multi-column tower, an additional 1/100 wind tunnel section model test for a cross-sectional width of 7 m was carried out to investigate the effect of the distances between the four columns on the value of the mean wind force coefficients. The distances between the centers of the columns are set to 25 mm and 55 mm, respectively. The wind tunnel section model setup for the 25 mm distance and 55 mm distance is shown in Fig. 6 and Fig. 7, respectively. The diameter of each column is 15 mm, and the length of each column is 1400 mm. The length of each column was selected so as to be long enough to reduce the wind tunnel blockage effect that can be caused by shorter distances between walls to which the model is attached. The wind tunnel section model was scaled to correlate with the cross-sectional shape of the multi-column tower. A three-component load cell was used to measure the forces and moments located at the end of the small-scale model.



Fig. 6 Wind tunnel section model setup for a 25 mm distance between columns.



Fig. 7 Wind tunnel section model setup for a 55 mm distance between columns.

2.2. Wind Tunnel Test Result

For the experimental models, the Reynolds number at the height of 0.25 m is 1.5x10 for the velocity of 10 m/s. The Reynolds number is calculated by Eq. 5:

$$Re = \frac{UL}{\nu}$$

where

U = characteristic velocity

L = characteristic length

νa = kinematic viscosity

The mean, standard deviation, and variance wind force coefficients in the y-axis for the wind axes are shown in Table 1. The reference width and height for all three towers are set to 0.023 m and 0.4 m, respectively. They are also used for calculating the mean wind force coefficients from the CFD simulation of the small-scale models for a fair comparison between the wind tunnel test and CFD simulation results. However, for the real-scale models, the exact projected area of each tower is considered in the calculation of the mean wind force coefficients.

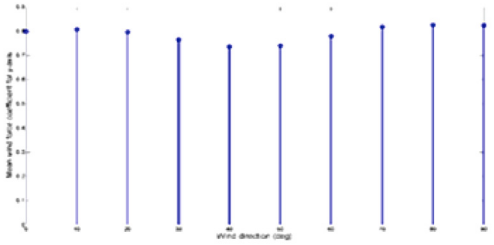


Fig. 8 Mean wind force coefficients for a multi-column tower for wind axes.

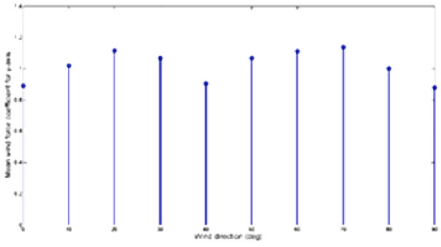


Fig. 9 Mean wind force coefficients for a modular tower for wind axes.

Table 1. Mean, standard deviation, and variance wind force coefficients at the y-axis for wind axes.

	Circular	Multi-column		Modular
	0 degrees	0 degrees	40 degrees	0 degrees
Mean	0.80	0.80	0.74	0.90
Standard deviation	0.06	0.05	0.15	0.17
Variance	0.002	0.001	0.009	0.016
Force (N)	0.45	0.45	0.42	0.51

It is shown that the modular tower with an octagonal cross section has the highest wind force coefficients, and the multi-column tower and conventional circular tower have the same value of 0 degrees for the wind force coefficients. However, the multi-column tower has the lowest wind force coefficient of 0.74 at 40 degrees. Therefore, the shape of the multi-column tower is shown to be effective in reducing the mean wind loads. For the multi-column tower and modular tower, the wind force coefficients in the y-axis for the wind axes are shown in Fig. 8 and Fig. 9, respectively. In Fig. 8, it is shown that the mean wind force coefficients at 0 and 90 degrees are slightly different due to experimental errors even though the multi-column tower is symmetric about 90 degrees. For the circular tower, the wind force coefficient for the wind axes is invariant with the wind direction, and therefore in any direction, its value is 0.80. The wind force and moment coefficients for each rotation angle θ for the multi-column tower and modular tower are shown in Fig. 10 and Fig. 11, respectively.

The wind tunnel section model test results for two different distances between the columns, 25 mm and 55 mm, are given in Table 2. It is shown that the mean wind force coefficient values are decreasing as the distance between the columns is decreasing. Under a 15 m/s uniform wind flow, the mean wind force coefficients are decreased by 8.54% as the distance between the columns decreases. It is shown that for the configuration of the three circular

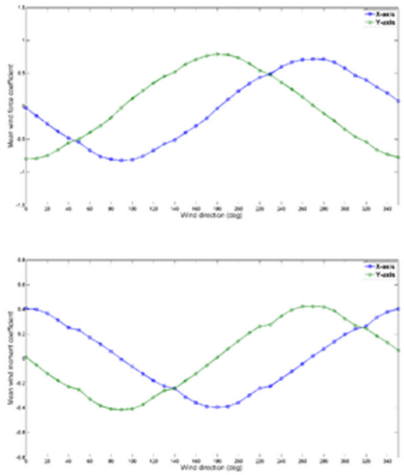


Fig. 10 Mean wind force and moment coefficients for a multi-column tower for structural axes.

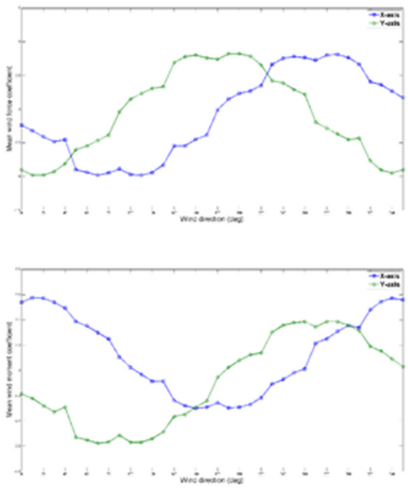


Fig. 11 Mean wind force and moment coefficients for a modular tower for structural axes.

cylinders in an equilateral-triangular arrangement, the mean wind force coefficients for each circular cylinder are increasing as the spacing ratio (the distance between the centers of the cylinders divided by the diameter of a cylinder) is increasing (Pouryoussefi et al., 2009).

In the wind tunnel test, the test setup, in which a three-component load cell was attached to the end of the wind tunnel section model, was designed to measure the total forces applied to each member subjected to the wind flow. To calculate the forces acting on each member rather than the sum of the forces acting on each member, an additional 2D CFD simulation was conducted for the wind tunnel section model. A detailed description of the 2D CFD simulation for the wind tunnel section model described above is given in the next section.

Table 2. Mean wind force coefficients for the wind tunnel section model for different distances between columns(55 mm and 25 mm).

	12 m/s	15 m/s
55 mm	1.15	1.17
25 mm	1.09	1.07

3_CFD SIMULATION

In this section, the CFD simulation was carried out for 1/300 small-scale models and real-scale models to obtain the mean wind force coefficients. The CFD simulation results were verified by the wind tunnel test results. For the real-scale models, the Reynolds number is increased dramatically due to the change of scale compared to that for the small-scale models. For the velocity of 10 m/s, the Reynolds number for the two models is 4.7×10^6.

During the simulation, the wind velocity for the small-scale models is set to 10 m/s, and the wind velocities for the real-scale models are set to 10, 30, 50, and 70 m/s. In this study, the finite volume method of the commercial software ANSYS CFX v13.0 was used for the discretization of the Navier-Stokes equations with the Shear Stress Transport (SST) turbulence model (ANSYS, 2010). The SST model is effective due to the use of the k–ω model in regions near the walls and the k–ε model in regions away from the walls.

Additionally, the 2D CFD simulation of the wind tunnel section model was conducted to calculate the forces on each column. The wind velocity during the simulation is set to 12 m/s and 15 m/s. The distances between the centers of the columns are set to 25 mm and 55 mm, respectively. To verify how the mean wind force coefficients of each column vary depending on the distances between the columns, the reference mean wind force coefficients for a single column were also calculated.

3.1. Computational Models

Figure 12 shows the computational domain and mesh used in the simulation. It is shown that the wind velocity is applied to the x-axis that is perpendicular to the left plane of the domain. The mesh is selected to be the Hexahedral type, and the computational domain (360 m × 120 m × 240 m) is set to be large enough to minimize the effect of the boundary conditions. Basically, the mesh (20 m × 20 m × 180m) around the structure was designed to be dense enough to guaranteea high accuracy off low prediction, such as the velocity, streamline, pressure change, and vortices. Furthermore, the value of the dimension less wall distance y+ is required to be below 1 since a finer mesh is used when the mesh is more closely around the structure in the computational domain (20 × 20 m × 180 m).

Table 3. Mean wind force coefficients and forces at the x–axis from the wind tunnel test and CFD simulation for small–scale models.

		Circular tower	Multi–column tower	Modular tower
Wind tunnel test	Mean	0.80	0.80	0.90
	Force (N)	0.45	0.45	0.51
CFD simulation	Mean	0.73	0.72	0.78
	Force (N)	0.42	0.41	0.44

shown in Table 4. In the calculation of the mean wind force coefficients, the exact projected area of each tower is considered. It is shown that the mean wind force coefficients for the multi-column type are smaller than those for the circular and octagonal types. Due to the difference in the Reynolds numbers for the wind tunnel test models and real-scale models, the mean wind force coeffi-

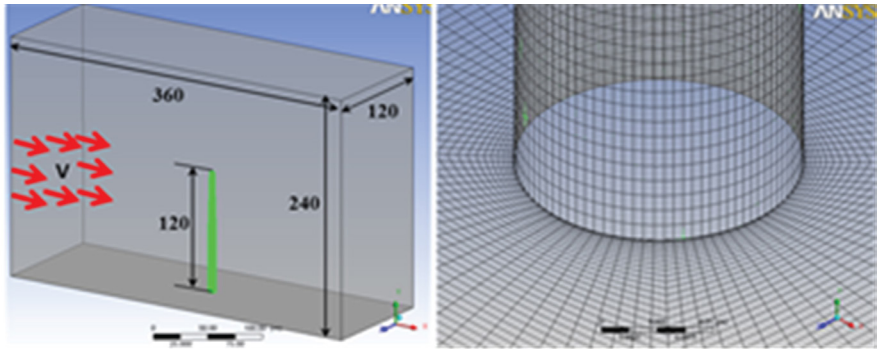


Fig. 12 Computational domain and mesh used for the CFD simulation.

3.2. Simulation Result

The mean wind force coefficients and forces from the wind tunnel test and CFD simulation are shown in Table 3. The reference width and height of the structure for calculating the mean wind force coefficients are set to 0.023 m and 0.4 m, respectively. The mean wind force coefficient values from the CFD simulation are shown to be lower than those from the wind tunnel test; the difference is within 10% for the circular tower and multi-column tower and slightly above 10% for the modular tower. In consideration of the experimental and numerical errors, it is believed that the results from the wind tunnel test and CFD simulation are reliable.

For the real-scale models, the mean wind force coefficients are cients do not coincide. It is generally known that for the circular cross section, the mean wind force coefficient decreases as the Reynolds number increases (Schewe, 1983; Holmes, 2001). In this study, the Reynolds number is 1.5×10^4 for the wind tunnel test model and 4.7×10^6 for the real-scale model. Therefore, the results for the wind tunnel test and CFD simulation seem to be consistent with the general tendencies of the effect of the Reynolds number on the wind force coefficients.

In contrast, the modular tower with an octagonal cross section is shown to be independent of the Reynolds number. Given that the Reynolds number is the ratio of the inertial forces to the viscous forces in the flow, it is generally known that sharp-edged bodies are

much less dependent on the Reynolds number due to their lower dependence on the viscous forces in the flow (Holmes, 2001). It is believed that for the real-scale models, the validation of the CFD model for the circular tower can be done through comparison with previously published results, as described above. For the modular tower with a sharp-edged octagonal cross section, it is believed that the validation of the CFD model can be done through a wind tunnel test due to the tower's lower dependence on the Reynolds number.

Table 4. Mean wind force coefficients at the x-axis from the CFD simulation for real-scale models.

	Circular tower	Multi-column tower	Modular tower
10 m/s	0.37	0.32	0.73
30 m/s	0.32	0.26	0.73
50 m/s	0.30	0.25	0.74
70 m/s	0.30	0.25	0.76

The velocity and pressure for the circular tower, modular tower, and multi-column tower at the height of 104 m are shown in Fig. 13. A wide spread of the wake region of the octagonal type is shown to increase the wind force coefficient compared to the other two types. The pressure distribution is shown representatively for only the velocity of 70 m/s. For different wind speeds, the pressure distribution is almost the same as that for the velocity of 70 m/s; the highest pressure value is located at the stagnation point, and the lowest pressure value is located where the velocity is increasing. The maximum pressure difference exists from the stagnation point to the wake region, and it was 105, 940, 2600, and 5000 Pa for the velocities of 10, 30, 50, and 70 m/s, respectively.

For the wind tunnel section model with 25 mm spacing and 55 mm spacing between columns, the mean wind force coefficients from the wind tunnel test and CFD simulation are shown in Table 5. It is shown that the difference between the values is within 5%. It is also shown that the CFD simulation results are consistent with the wind tunnel test results in that the mean wind force coefficient values are decreasing as the spacing between the columns is decreasing. In Fig. 14, the pressure, velocity contour, and turbulence kinetic energy plots for a single column, columns with 25 mm spacing, and columns with 55 mm spacing are shown. The mean wind force coefficients for each column from the CFD simulation are shown in Table 6. Due to the symmetrical arrangement of the

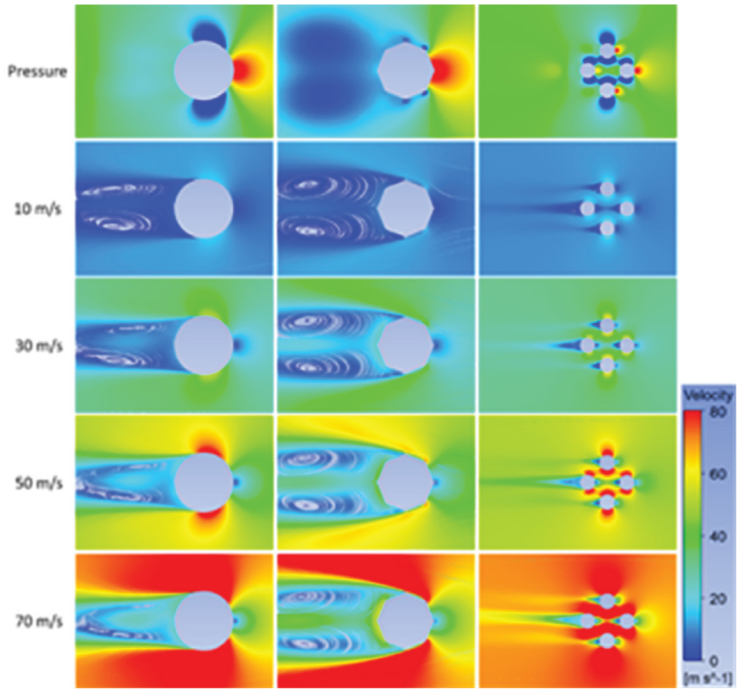


Fig. 13 Pressure and velocity contour plots for a circular tower, a modular tower, and a multi-column tower.

four columns as shown in Fig. 14, the mean wind force coefficients of the two columns located in the middle of the other two columns have identical values. The mean wind force coefficients of only three columns are shown in Table 6. It is shown that for the columns with 55 mm spacing, the increase in the values of the mean wind force coefficients is due to the increase in the value of the mean wind force coefficient for column 2, compared to the columns with 25 mm spacing.

Table 5. Mean wind force coefficients from the wind tunnel test and CFD simulation for the wind tunnel section model for different distances between columns (25 mm and 55 mm).

	25 mm		55 mm	
	Wind tunnel test	CFD simulation	Wind tunnel test	CFD simulation
12 m/s	1.09	1.06	1.15	1.20
15 m/s	1.07	1.03	1.17	1.17

4_CONCLUSIONS

In this study, a wind tunnel test and a CFD simulation for small-scale wind turbine tower models were carried out by the Daewoo Institute of Construction Technology in Korea. For a circular tower, a modular tower, and a multi-column tower, the reliability of the wind tunnel test and CFD simulation for 1/300 scale models was verified. Furthermore, a CFD simulation for real-scale models was carried out that considered the Reynolds number effect. For the multi-column tower, an additional 1/100 wind tunnel section model test was carried out to investigate the effect of the distance between the four columns, and the wind tunnel test results were compared with the results from the 2D CFD simulation.

From a comparison of the results from the wind tunnel test and CFD simulation for the 1/300 small-scale models, it is believed

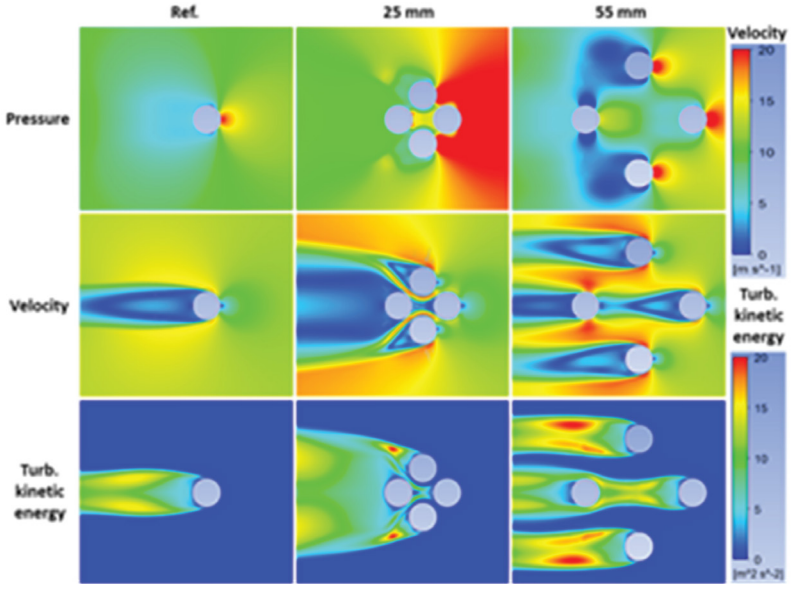


Fig. 14 Pressure, velocity contour, and turbulence kinetic energy plots for a single column, columns with 25 mm spacing, and columns with 55 mm spacing.

Table 6. Mean wind force coefficients of three columns from a CFD simulation for the wind tunnel section model for different distances between columns (25 mm and 55 mm).

	Ref.	25 mm			55 mm		
		Col. 1	Col. 2	Col. 3	Col. 1	Col. 2	Col. 3
12 m/s	0.83	0.97	0.92	0.37	0.93	1.14	0.40
15 m/s	0.82	0.96	0.88	0.35	0.93	1.12	0.35

that the results are reliable in consideration of the experimental and numerical errors. The difference between the values was shown to be within 10% for the circular tower and multi-column tower. For the modular tower, it was slightly above 10%. For the wind tunnel section model, the difference between the values from the wind tunnel test and 2D CFD simulation was shown to be within 5%.

It was also shown that the force coefficients obtained from the real- scale models through the CFD simulation have smaller values than the force coefficients obtained from the small-scale models through the wind tunnel test due to the Reynolds number effect; the Reynolds number is 1.5×10^4 for the small-scale models and 4.7×10^6 for the real-scale models. For the circular tower, the force coefficient obtained from the wind tunnel test is 0.80, and the force coefficient obtained from the CFD simulation is 0.30 ~ 0.37. It is generally known that the flow patterns around a circular shape are highly dependent on the Reynolds number; thus, a change in the flow patterns results in changes in the wind force and moment coefficients. For the multi-column tower, it was shown that the mean wind force coefficient values are decreasing as the distance between the columns is decreasing.

In this study, the reliability of the wind tunnel testing and CFD simulation for the newly proposed steel wind turbine towers was discussed and the CFD simulation for the real-scale models was carried out. Further research on the loading effect of the blades and nacelle on the wind turbine towers will be done, and additional cross-sectional shapes for a wind turbine tower will be considered.

ACKNOWLEGEMENTS

This research was supported by a grant (Code 12 Technology Innovation E09) from the Construction Technology Innovation Program funded by the Ministry of Land, Infrastructure and Transport (MOLIT) of the Korean Government.

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흙도로포장용 재료로서 폐콘크리트 미분말의 활용성 연구

Utilizability of waste concrete powder as a material for soil pavement

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본 논문은 2015년 한국건설순환자원학회 논문집, Vol.3, No.3, pp. 277-282에 게재되었던 논문을 재게재한 것입니다.

Abstract

This study is conducted to utilize waste concrete powder (WCP) made as a by-product manufacturing high quality recycled aggregate. The blaine fineness of the used waste concrete powder was 928 m^2/g. As the main characteristic of waste concrete powder, it showed an angular type similar to cement, but hydrated products were attached on the surface of particles. In addition, the size of the particles of waste concrete powder was larger than OPC and in terms of chemical components it had higher SiO_2 contents. For using WCP in soil cement-based pavement, the qualities, physical and chemical properties, of WCP should be researched. In the first step, the specified compressive strength of mortar for two types of clay sand soil and clay soil respectively was experimented to be 15 MPa and then optimum mixing ratio of chemical solidification agent were decided in the range of 1.5 - 3.0 % in the replacement with cement weight content. In the second step, based on the prior experimental results, recycling possibility of WCP in soil cement-based pavement was studied. In the result of experiment the mixing ratio of WCP were 5, 10, 15 and 20 % in the replacement with soil weight and the compressive strength of mortar was somewhat decreased according to the increase of the mixing ratio of WCP.

Keywords : CFT truss girder, Negative bending moment, Continuous composite member shape

1_연구의 목적

최근 경제발전에 힘입어 생활수준이 크게 향상됨으로써 이로 인하여 발생되는 쓰레기, 생활 및 산업 폐기물이 크게 증가하여 심각한 사회문제로 대두되고 있다. 특히, 산업 폐기물 중 건설폐기물은 콘크리트구조물의 노후화, 성능저하 및 용도, 목적의 변경 등으로 인하여 점차 증가되고 있는 실정이다.

이러한 건설폐기물 중 약 65%를 차지하고 있는 폐콘크리트는 현재까지 국내의 경우 단순히 성토나 매립용 재료로 90%이상 주로 사용 내지는 폐기되어 왔을 뿐, 도로 기층이나 콘크리트 레미콘용 등의 부가가치가 높은 자원으로서 재활용되지 못하고 있으며, 더구나 이러한 건설폐기물을 처리할 수 있는 매립지의 절대 부족에 따른 폐기물의 처리비용 상승이 큰 부담이 되고 있다(Ahmad 2003, Berndt 2009). 폐콘크리트를 이용한 재생골재를 보다 부가가치가 높은 도로 기층용이나 콘크리트용과 같이 천연골재와 대체하여 사용할 수 있는 고품질의 재생골재를 생산하기 위해서는 파쇄 및 처리 과정이 여러 번인 정밀한 제조 기술이 필요하다. 이러한 제조 기술로 생산된 재생골재는 표면에 부착된 모르타르를 최대한 제거할 수 있어 고품질의 재생골재를 생산할 수 있다(Kim 2012, Moon 2002). 그러나 이러한 제조 기술은 재생골재 자체의 품질을 향상시킬 수 있지만, 제조원가 상승 및 폐콘크리트 분말의 발생량을 크게 증가시키는 문제점이 있다. 현재로서는 이때 발생되는 폐콘크리트 분말을 전량 점토와 혼합하여 단순 매립하고 있다. 그러나 최근 들어서 매립지 확보의 어려움과 토양 및 지하수 오염등과 같은 환경적인 측면이 문제점으로 지적

되고 있다(Jiusu 2009).

이러한 문제점을 해결하기 위해서 몇몇 연구자들은 폐콘크리트 분말을 콘크리트용 혼화재료, 지반 개량재 및 담채 등으로 적용하는 연구를 수행하고 있다(Tsung 2006).

한편, 현재까지 국내의 도로포장은 대부분이 표층에 불투수층인 아스팔트 및 시멘트 콘크리트를 사용한 포장으로 노상과 표층이 차단됨에 따라 노상의 흙은 점진적으로 산성화가 진행되고 이에 따라 영양물질이 부족 하여 미생물이 서식하지 못하게 될 뿐만 아니라 토양의 건조화가 진행되어 여러 가지 환경문제가 발생하고 있는 실정이다. 따라서 표층재료를 자연상태의 흙을 사용한 흙포장 도로는 기존의 아스팔트 및 시멘트 콘크리트 포장과는 다르게 흙, 무기질계 특수혼화재료, 골재, 혼합수를 최적배합으로 하여 중·저강도 구조물이다. 또한, 공법개선을 통하여 포장단면 변화를 조절하므로써 경제적으로 시공이 가능하며, 자연상태의 흙을 주원료로함으로써 친환경적이며, 주변환경과 조화를 이룰 수 있는 포장이라 할 수 있다.

따라서 본 연구에서는 폐콘크리트 분말을 흙도로포장용 충전재료로서 사용하기 위한 가능성을 검토하였다.

2_실험개요

2.1 사용재료

2.1.1 시멘트 및 결합재의 품질

시멘트는 밀도 3.15 g/cm^{3}인 보통포틀랜드 시멘트(OPC)를 사용하였으며, 고로슬래그 미분말(GGBF)은 밀도 2.90 g/cm^3, 폐콘크리트 미분말(WCP)은 밀도 2.48 g/cm^{3}인 것을 사용하였다. Table 1은 시멘트 및 결합재의 화학성분 및 물리적 성질을 정리한 것이다.

2.1.2 고화제 및 토양의 품질

고화제는 석고 및 아윈 성분으로 구성된 Road Compound(RC)와 탄산소다 및 알루미늄설페이트로 제조된 New Road Compound(NRC)를 사용하였다. 또한, 본 실험에 사용된 대상토는 최대건조밀도 2.014 g/cm^3 및 최적함수비 17%인 사질토를 사용하였다. Table 2는 고화제 및 사질토의 화학성분을 정리한 것이다.

Table 1. Chemical components and physical properties of OPC and WCP

Items	OPC*	GGBF**	WCP***
SiO_2 (%)	21.95	34.3	58.6
Al_2O_3 (%)	6.59	15.8	10.4
Fe_2O_3 (%)	2.81	0.1	4.6
CaO (%)	60.10	42.2	11.8
MgO (%)	3.32	6.8	1.2
SO_3 (%)	2.11	0.2	0.4
Density (g/㎤)	3.15	2.90	2.48
Blaine Fineness (㎠/g)	3,112	4,380	948

* OPC : Ordinary Portland Cement
** GGBF : Ground Granulated Blast-Furnace Slag
*** WCP : Waste Concrete Powder

Table 2. Chemical components of hardening agent

Items	RC*	NRC**	Sandy
SiO_2 (%)	8.5	5.3	65.7
Al_2O_3 (%)	15.2	33.0	17.9
Fe_2O_3 (%)	1.4	1.0	2.1
CaO (%)	39.7	12.0	0.3
MgO (%)	1.1	1.0	0.5
Na_2O (%)	0.1	5.0	0.4
SO_3 (%)	33.4	42.2	–
K2O (%)	–	–	5.2
Ig. Loss	0.6	0.5	7.7

* RC : Road Compound
** NRC : New Road Compound

2.2 실험방법

2.2.1 미세구조분석

OPC에 RC 또는 NRC를 혼합한 시멘트 페이스트를 분쇄하여 150㎛ 체에 통과시켜 채취한 시료에 대하여 RINT D/max 2500 (Rigaku)으로써 X선회절분석을 실시하였으며, 측정조건은 CuKα (Ni filter): 30 kV, 20 mA, Scanning speed: 2 °/min, Full scale 7000 cps, 2θ : 5 ° ~ 40 ° 로 정하였다. OPC에 RC 또는 NRC를 혼합한 시멘트페이스트의 초기 재령 3 및 7일에서 어떠한 수화생성물이 관찰되는지 알아보기 위하여 10,000 배로 확대하여 주사전자현미경(SEM)으로 촬영하였다.

2.2.2 역학적 성능 검토

압축강도용 공시체는 KS L 5105에 의하여 시멘트, 사질토, 혼합수 및 RC 또는 NRC를 혼합하여 50×50×50 mm

인 입방체 몰드를 사용하여 강재 다짐봉에 의해 다짐의 횟수를 균일하게 한 후 2층 다짐을 실시하여 공시체를 제작하였다. 압축강도 측정은 KS F 2514에 의하여 측정하였다.

2.2.3 배합

사질토에 대한 Road Compound의 적정 혼합률 및 최적 함수비 결정을 위한 배합은 사전 배합에서 얻어진 결과를 분석하여 사질토에 대한 함수비 혼합률 4수준(10%, 15%, 20%, 25%), 사질토에 대한 Road Compound의 혼합률 30%, Road Compound에 대한 RC 혼합률(20%, 30%)으로 변화시켜 제조하였다. 배합표는 Table 3과 같다. Table 4는 흙도로 포장에서 가장 큰 비용을 차지하는 NRC의 혼합률을 경제성 측면을 고려함으로써 최적의 NRC 혼합률을 결정하기 위하여 NRC/OPC+NRC를 4수준(1.5, 3, 4.5, 6%)으로 혼합률을 변동하여 실험을 실시하였다.

Table 3. Mixture proportions of mortar (I)

MC* (%)	(OPC+RC)/Soil(%)	RC/(OPC+RC)(%)	Unit mass (kg/m³)			
			W	OPC	RC	Soil
0	30	20	197	440	110	1,950
15			285	440	110	
20			373	440	110	
35			461	440	110	

* MC : Moisture Content (%)

Table 4. Mixture proportions of mortar (II)

(OPC+NRC)/Soil (%)	NRC/(OPC+NRC) (%)	Unit mass (kg/m³)			
		W	OPC	RC	Soil
10	1.5	302	198	2.97	2,014
15			298	4.54	
10	3.0		195	6.04	
15			293	9.06	
10	4.5		192	9.06	
15			299	13.6	
10	6.0		189	12.08	
15			284	18.12	

3_실험결과 및 고찰

3.1 미세구조분석

고화재 RC 및 NRC가 시멘트 및 혼합수와 반응하여 어떤 수화생성물을 생성하는지 알아보기 위하여 보통포틀랜드시멘트(OPC)와 OPC에 RC 또는 NRC를 각각 혼합한 시멘트 페이스트를 분쇄하여 150 ㎛ 체에 통과시켜 채취한 시료에 대하여 재령별로 X선 회절 분석을 실시한 결과를 Fig. 1에 나타내었다. Fig. 1에서 재령 및 고화재 종류에 관계없이 에트린가이트(E), 포틀랜다이트(P), 및 칼사이트(C) 등이 관찰되었다. 특히, OPC+RC를 혼합한 시멘트 페이스트의 경우 에트린가이트(E)가 다른 페이스트의 경우보다 크게 나타나는 경향을 보였다. 이러한 원인은 RC에 들어있는 아윈 및 무수석고와 같은 팽창제 성분이 초기에 물과 반응하여 많은 양의 에트린가이트를 생성하였기 때문으로 판단되며 침상구조의 에트린가이트가 흙 내부의 큰 공극을 채워줌으로써 흙입자 사이의 충분한 결합력을 발현하게 되는 것으로 판단된다. 또한, 경제성을 고려하여 RC의 대체 고화재로 사용하고자 하는 NRC를 혼합한 OPC+NRC의 경우에는 OPC+RC를 혼합한 경우와 마찬가지로 강도를 발현시킬 수 있는 에트린가이트 및 포틀랜다이트를 생성하는 것을 알 수 있었다. Fig. 2는 각 재령별로 보통포틀랜드시멘트 (OPC), OPC에 RC 및 NRC를 각각 혼합한 페이스

트의 SEM사진을 나타낸 것이다. 고화재를 혼합하지 않은 시멘트페이스트의 경우 재령 3일에서는 에트린가이트가 선명하게 나타나고 있지만, 재령 7일에서는 수산화칼슘이 성장하여 대부분을 차지하고 있었다. 한편, OPC+RC의 경우 재령에 관계없이 침상구조인 에트린가이트가 대부분을 차지하고 있으며, OPC+NRC의 경우에는 재령 3일에서 판상의 NAH(Sodium-Aluminate-Hydrogen)로 추정되는 물질이 형성됨을 알 수 있었다.

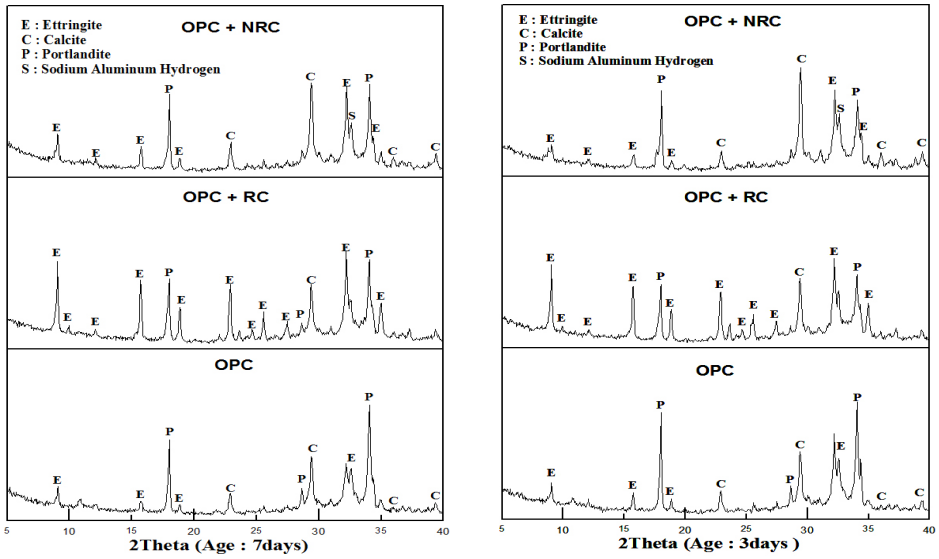
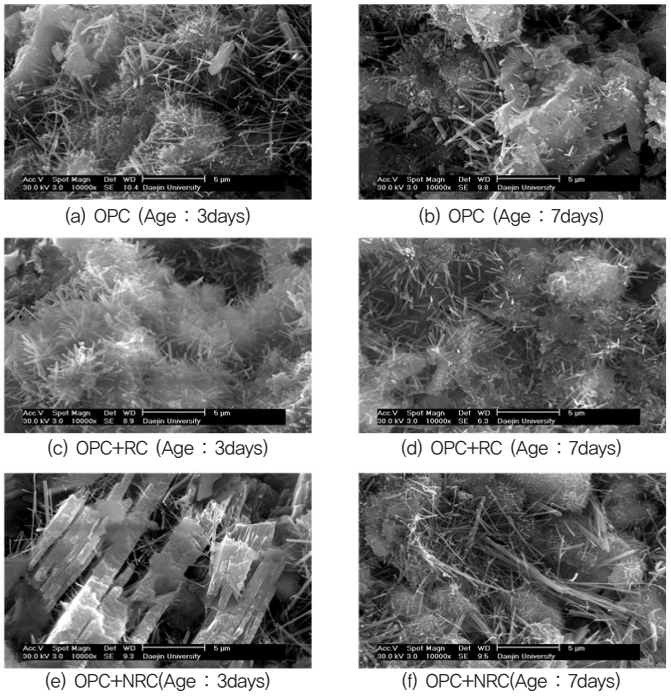


Fig. 1. X-ray diffraction of paste



(a) OPC (Age : 3days) (b) OPC (Age : 7days)

(c) OPC+RC (Age : 3days) (d) OPC+RC (Age : 7days)

(e) OPC+NRC(Age : 3days) (f) OPC+NRC(Age : 7days)

Fig. 2. SEM of paste

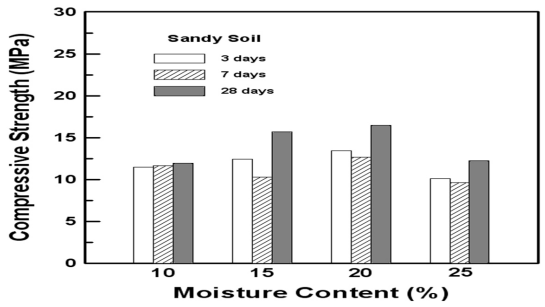


Fig. 2. 4-Points bending test on CFT

3.2 압축강도 특성

사질토에 Road Compound의 혼합률 30%, RC 혼합률 20%로 고정하고 강도특성을 고려한 최적의 함수비를 결정하기 위하여 함수비 혼합률 4수준(10%, 15%, 20%, 25%)으로 변화시킨 공시체를 제조하였다. 제조한 공시체의 재령 3일, 7일 및 28일의 압축강도를 측정고 함수비 변화에 따른 압축강도 값을 나타낸 것이 Fig. 3 이다. Fig. 3의 결과 사질토는 함수비 20%에서 재령 28일의 강도가 가장 크게 나타나고 있음을 알 수 있다. 그러나 초기재령인 3일 및 7일에서 함수비 변화에 따른 강도차이는 크지 않으며, 재령 28일의 함수비 20%에서 강도를 기준으로 할 경우 함수비 15%에서는 약 10% 압축강도가 다소 감소되었지만, 함수비 25%에서는 약 22%로 압축강도가 크게 감소하고 있음을 알 수 있다. 이러한 결과는 시험에 사용한 사질토의 최적함수비(OMC)가 17%로 Road Compound를 사용한 사질토의 강도가 가장 크게 증가된 함수비 20%와 유사한 값을 나타내고 있음을 알 수 있어 사질토에 Road Compound를 사용시 강도특성을 고려한 최적의 함수비는 사용 사질토의 최적함수비(OMC) 또는 최적함수비(OMC)보다 함수비를 1 ~ 3% 정도 다소 증가시킨 함수비로 결정하는 것이 적당하다고 판단된다. Fig. 4 및 5는 NRC의 최적 혼합률을 알아보기 위해 정리한 것이다. 사전의 실험결과를 바탕으로 최적의 함수비 15%, 흙대비 시멘트량 10 및 15% 각각에 대하여 시멘트 대비 NRC 혼합률을 0, 1.5, 3.0, 4.5 및 6.0% 5단계로 혼합하여 모르타르의 압축강도를 재령 3, 7 및 28일에 대하여 나타낸 것이 Fig. 4 및 5 이다. Fig. 4 및 5에서 알 수 있듯이 재령 및 NRC 혼합률에 관계없이 OPC+NRC/Soil이 10 및 15% 모든 경우에 압축강도 15 MPa를 상회하는 결과를 얻었다. 특히, 시멘트량이 15%인 경우에는 시멘트의 혼합률이 증가함에 따라 시멘트의 수화반응이 활발하게 일어나 비교적 높은 압축강도를 얻었다. 반면에 NRC의 혼합률이 증가에 따라 압축강도의 증가가 비례하여 증가하는 것이 아니기 때문에 목표로 하는 적당한 강도 및 경제성을 고려하여 NRC/OPC +NRC 가 1.5 및 3.0%를 혼합하는 것이 바람직할 것으로 판단된다. Fig. 6은 고로슬래그미분말을 0, 15, 30 및 45% 혼합하여 실험한 결과를 고려하여 고로슬래그미분말을 30%로 고정한 후 폐콘크리트분말을 흙 대체 0 ~ 15% 까지 5%씩 증가하여 실험한 결과를 나타

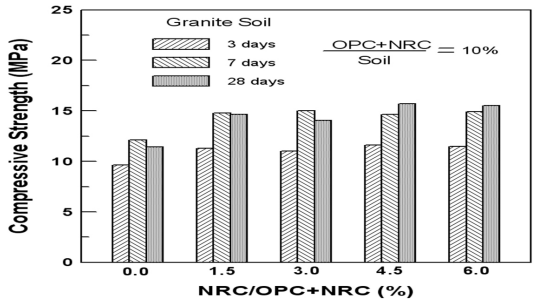


Fig. 4. Compressive strength according to NRC (10%)

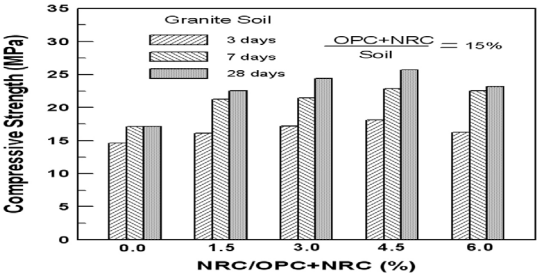


Fig. 5. Compressive strength according to NRC (15%)

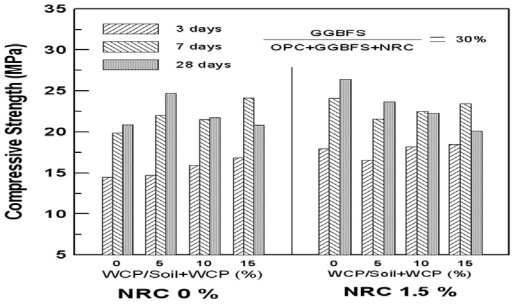


Fig. 6. Compressive strength according to WCP

낸 것이 Fig. 6 이다. Fig. 6에서 폐콘크리트분말의 혼합률에 관계없이 NRC의 혼합률 0 및 1.5%에 대하여 압축강도 15MPa를 상회하는 결과를 얻을 수 있었다. 따라서 폐콘크리트분말의 최대 혼합률을 15%까지 혼합하여 사용하는 것이 바람직한 것으로 생각된다.

4_결 론

본 연구의 결과를 정리하면 다음과 같다.

1) 사질토의 최적함수비(OMC) 전후로 함수비(MC)를 변화시켜 실험한 결과 최적함수비를 약간 상회하는 15 ~ 20%에서 가장 좋은 압축강도가 나타남을 확인할 수 있었으며, 본 연구에서 흙포장도로에 사용된 고화제인 Road Compound(RC) 대체 재료로서 New Road Compound(NRC)가 충분히 가능함을 확인하였다.
2) New Road Compound에 의한 사질토의 최적함수비(OMC) 전후로 함수비(MC)를 변화시켜 실험한 결과, 최적함수비를 약간 상회하는 혼합률에서 가장 좋은 압축강도가 나타남을 확인할 수 있었으며, 본 연구에서 흙포장도로에 사용된 고화재인 Road Compound 대체 재료로서 New Road Compound가 충분히 가능함을 확인하였다.
3) New Road Compound를 혼합한 경우, 경제성을 고려하여 시멘트에 고로슬래그미분말을 0, 15, 30 및 45% 4단계로 대체하여 실험한 결과 최적의 혼합률이 30%로 나타났으며, 폐콘크리트분말의 경우 흙에 대하여 최대 15%까지 혼합하여 사용하여도 충분히 목표강도를 확보할 수 있었다.

이상의 연구결과를 통하여 개발된 Road Compound 및 New Road Compound로써 표층흙을 고화시킬 목적으로 사용할 경우 압축강도를 5 MPa에서 최대 29 MPa정도까지 발현시킬 수 있으므로 흙포장 도로의 사용목적에 따라 즉 압축강도가 12 ~ 15 MPa 범위에서는 보도 및 자전거도로용, 15 ~ 18 MPa 범위에서는 경교통로 및 주차장용, 18 ~ 25 MPa 범위에서는 일반도로용으로 사용이 가능할 수 있을 것으로 판단된다. 또한, Road Compound 및 New Road Compound의 혼합률을 조정하면 지반개량공사에 있어 연약지반 개량 및 안정처리용 등으로 폭 넓게 활용할 수 있을 것으로 판단된다.

감사의 글

이 논문은 국토해양부 건설기술혁신사업의 연구비 지원(11기술혁신 F04-탄소저감형연구단)에 의해 수행되었습니다.

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흙도로포장용 재료로서 폐콘크리트 미분말의 활용성 연구

본 연구는 고품질의 재생골재 제조시 발생하는 폐콘크리트 미분말을 콘크리트용 혼화재료로 활용하기 위한 연구이며, 분말도는 928 cm2/g인 폐콘크리트 미분말에 대하여 검토하였다. 폐콘크리트 분말의 주요 특징은 시멘트와 유사한 각진 입형을 나타내고 있었으나 입자 표면에 수화생성물들이 부착되어 있었다. 또한 시멘트와 비교하여 폐콘크리트 분말의 입자 크기는 크게 나타났으며, 화학성분은 SiO2 함량이 높게 나타났다. 한편, 도로포장은 대부분이 표층에 불투수층인 아스팔트 및 시멘트 콘크리트를 사용한 포장으로 노상과 표층이 차단됨에 따라 노상의 흙은 점진적으로 산성화가 진행되어 영양물질이 부족하여 미생물이 서식하지 못하게 될 뿐만 아니라 토양의 건조화가 진행되어 여러 가지 환경문제가 발생하고 있다. 연구 결과 개발된 Road Compound 및 New Road Compound로써 표층흙을 고화시킬 목적으로 사용할 경우 압축강도를 5 MPa에서 최대 29 MPa정도까지 발현시킬 수 있으므로 흙포장 도로의 사용목적에 따라 즉 압축강도가 12 ~ 15 MPa 범위에서는 보도 및 자전거도로용, 15 ~ 18 MPa 범위에서는 경교통로 및 주차장용, 18 ~ 25 MPa 범위에서는 일반도로용으로 사용이 가능할 수 있을 것으로 판단된다. 또한, Road Compound 및 New Road Compound의 혼합률을 조정하면 지반개량공사에 있어 연약지반 개량 및 안정처리용 등으로 폭 넓게 활용할 수 있을 것으로 판단된다.

요철형 이음단면을 갖는 프리캐스트 교량 바닥판의 휨성능 평가

Evaluation on Flexural Performance of Precast Bridge Decks with Ribbed Connection

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본 논문은 2015년 한국구조물진단유지관리공학회 논문집, Vol.19, No.3, pp. 1-9에 게재되었던 논문을 재게재한 것입니다.

Abstract

Due to the increasing number of deteriorated bridges worldwide, the importance of maintenance and replacement of existing bridges are being emphasized. Cast-in-place concrete deck which is mainly applied to deck replacement of existing bridges have problems such as deterioration concerns by initial crack, labor cost increase, difficulties of maintenance and replacement, construction time increase, and indirect cost increase by traffic congestion. On the contrary, a precast concrete deck is considered as an effective alternative because of its quality assurance and accelerated construction. The connection method ensuring the required strength and durability is especially important, because the connection part of the precast concrete deck is vulnerable to cracks and leakage. Therefore, this study proposes precast bridge decks with ribbed connection which are more improved than existing bridge deck joints, and flexural performance is verified through various parameter tests.

Keywords : Precast concrete deck, Accelerated construction, Ribbed connection, Flexural performance

1. 서 론

기존 교량 바닥판의 노후화로 인해 신설 및 교체 등 유지관리의 중요성이 증가되고 있다. 현재 바닥판 교체 및 신설에 주로 적용되고 있는 현장타설 바닥판은 인건비 상승, 보수 및 교체의 어려움, 공기지연, 부실시공 우려, 바닥판의 보수보강 및 교체시 교통체증으로 인한 간접비 증가 등 여러 문제점을 내포하고 있다. 그리하여 품질확보와 급속시공이가능한 프리캐스트 교량 바닥판 공법이 개발되어 적용이 확대되고 있다.

프리캐스트 교량 바닥판의 경우 바닥판간 이음부의 균열, 누수 등 사용성 문제가 가장 중요하다. 이음부 연속화를 위한 기존 공법으로 내부긴장재를 이용하여 이음부 사이에 압축력을 도입하는 방법, 겹이음을 이용한 방법, 헤드철근이나 나선철근 등을 이용하여 인장에 저항하는 방법, 후크형태의 루프철근을 이용하는 방법, 볼트나 용접 등을 이용하는 방법 등이 있다. 기존 프리캐스트 바닥판 공법 중 내부긴장재로 이음부를 연속화하는 공법은 구조성능 및 내구성은 우수하나 내부긴장 공정으로 경제성 및 시공성이 불리하며, 루프철근을 이용한 프리캐스트 바닥판은 경제성은 우수하나 이음부의 현장타설로 급속시공이 어려우며 균열, 누수 등의 우려 가 있다. 그리하여 기존 공



Fig. 1 Accelerated construction of precast bridge deck

법을 개선하여 소요강도 및 경제성을 만족하면서 급속시공이 가능한 프리캐스트 교량 바닥판의 이음부 연결기술의 개발이 필요하다.

2_기존 프리캐스트 교량 바닥판 이음부 연결기술

2.1 개요

프리캐스트 교량 바닥판은 공기단축과 콘크리트 품질확보 라는 장점 때문에 점차 시공이 확대되고 있지만, 이음부의 균열, 누수 등 사용성 문제로 프리캐스트 바닥판의 휨성능과 균열제어 성능을 개선하려는 연구가 지속적으로 수행되어 다양한 바닥판 이음부 연결기술이 제안되었으며, 도심지의 공기단축과 급속시공 및 부분교체에 대한 수요가 증가함에 따라 프리캐스트 구조물 관련기술에 대한 연구도 지속적으로 증가하고 있다 (DAEWOO, 2000). 기존의 프리캐스트 바닥판 이음부 연결기술은 다음과 같다.

2.2 내부긴장 이음

내부긴장 이음은 콘크리트 바닥판 내부에 긴장재를 삽입하여 바닥판끼리 연결한 후 텐던의 긴장 및 정착을 통해 압축력을 도입하여 바닥판 간 이음부의 연속화를 확보하는 방법이다 (Shim et al., 2001). 내부긴장 공법은 가장 일반적인 바닥판 연결방법으로서 프리캐스트 바닥판을 사용하는 대부분의 신설교량 및 바닥판 교체 시공에 사용되고 있다. 현재 사용되고 있거나 개발된 이음 방법 중 이음부의 구조성능 및 내구성이 가장 뛰어나므로 가장 널리 사용되고 있으나, 내부-긴장 공법의 가장 큰 단점은 경제성 확보가 어렵다는 것이다. 긴장재와 정착구 등 텐던 관련 공정의 추가로 공사비 증가의 주된 원인이 되며, 바닥판 총 공사비의 50%에 이르기도 한다. 신설시공 시에는 거더의 단면을 축소하여 전체 교량 공사비를 감소시킬 수 있으나, 바닥판 부분교체 공사의 경우 다른 바닥판 연결 방법에 비해 공사비가 훨씬 높기 때문에 내부긴장공법을 적용할 경우 경제성을 확보하기 어려운 면이 있다.

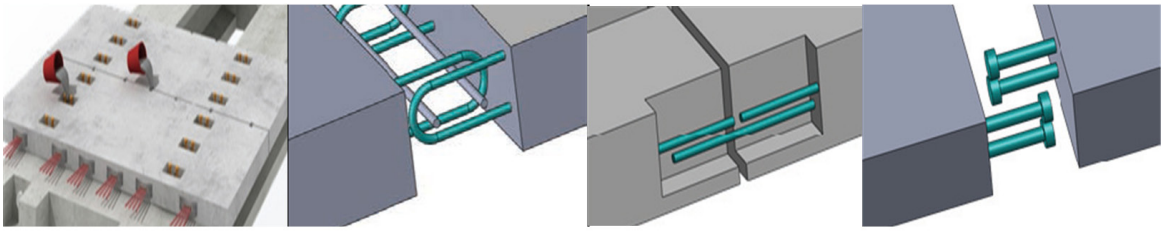
2.3 철근이음

철근이음은 직선 또는 일정한 형상을 가진 철근을 겹치게 배치한 후 현장에서 이음부에 콘크리트를 타설하여 바닥판간 연결성을 확보하는 방법이다. 철근이음에는 일반적인 겹이음이나 루프철근 이음, 헤드철근 이음 등 철근 형상에 따라 다양한 이음방식이 있다.

일반적인 겹이음은 주철근으로 사용되는 일반적인 직선철근을 길게 연장하여 일정길이 이상 겹친후 콘크리트를 타설하여 연결성을 확보하는 이음이다. 루프철근 이음은 일반적인 겹이음 방식과 동일하나, 겹이음에 사용되는 철근을 후크형태로 가공하여 철근이음 길이를 일반 겹이음보다 단축할 수 있다. 헤드철근 이음은 철근단부에 원형이나 사각형의 헤드를 부착하여 지압력을 통해 소요내력을 확보하여 철근이음 길이를 축소할 수 있다.

2.4 기존 기술 분석 및 개발 기술의 효과

프리캐스트 바닥판 이음부의 연속화를 위한 기존 연결기술 중 내부긴장 연결기술은 내부텐던을 이용하기 때문에 부분교체 및 유지관리가 어렵고, 텐던 공정의 긴장재 및 정착구 비용 추가로 공사비가 증가하고 작업난이도가 높다. 기존 루프철근 이용한 연결기술은 내부텐던이 없기 때문에 내부긴장 이음에 비해 경제적이고 시공성이 높은 반면, 이음부의 거푸집 설치와 콘크리트 타설 및 양생으로 공기가 증가하며 이음부의 균열, 누수 등 사용성 문제



(a) Post-tensioning (b) Loop joint (c) Lap splice (d)Headed reinforcement

Fig. 2 Existing precast deck connection

가 우려된다.

이러한 기존 프리캐스트 바닥판 연결기술의 한계를 개선하고자 소요강도와 경제성을 만족하면서 부분교체 및 급속시공이 가능한 프리캐스트 교량 바닥판의 이음부 연결기술에 대한 연구를 수행하게 되었다. 본 연구를 통해 기존 노후화교량 바닥판의 경제적인 부분교체 및 급속시공 기술의 개발이 가능할 것으로 사료된다.

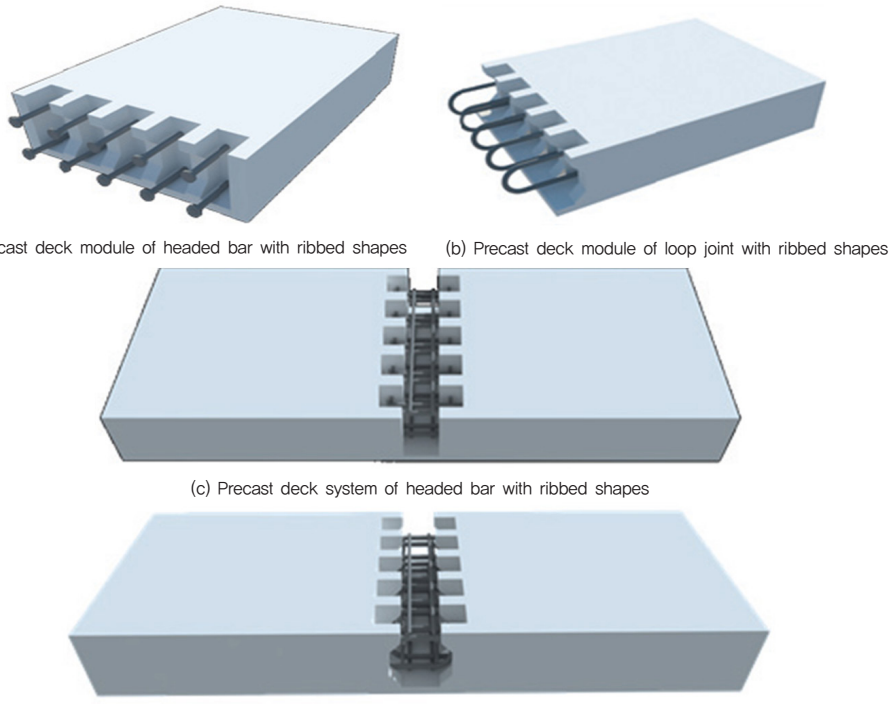
3_요철형 프리캐스트 콘크리트 바닥판 연결기술 제안

3.1 개요

앞서 언급하였듯이 기존에 개발된 프리캐스트 바닥판 공법 중 현재 가장 널리 사용되고 있는 내부긴장재로 이음부를 연속화하는 공법은 종방향 텐던 관련 공정의 추가로 경제성 및 시공성이 불리하며, 루프철근을 이용한 프리캐스트 바닥판 공법은 이음부의 현장타설 및 양생기간 증가로 급속시공이 어려운 단점이 있다. 그리하여 종방향 내부긴장재 없이 이음부의 무수축몰탈 타설로 급속시공이 가능한 프리캐스트 바닥판 이음부의 연결기술 개발을 위한 연구를 수행하였다. 그 결과 텐던 관련 비용의 절감 및 공기단축을 통해 경제성과 시공성을 개선할 수 있는 프리캐스트 바닥판 연결시스템을 제안하였고, 제안된 단면의 휨성능 검증을 위해 기존 연결기술을 포함하여 부재요소실험을 수행하였다.

3.2 요철형 프리캐스트 바닥판 모듈 및 연결시스템 제안

루프철근을 이용한 프리캐스트 바닥판 이음부의 휨성능은 부재요소실험을 통해 분석된 바 있다 (Chang et al., 2007). 본 연구에서는 총2회의 요소실험을 수행하였는데, 1차 요소실험 결과로부터 이음부의 철근겹이음 길이 확보, 이음부 횡철근 보강, 이음단면의 부착강도 향상을 위한 요철형 이음단면 적용 등 프리캐스트 바닥판 이음부의 연결상세를 개선하였다. 그 결과 Fig. 3과 같이 요철형 이음단면을 갖는 프리캐스트 바닥판 모듈 및 연결시스템를 제안



(a) Precast deck module of headed bar with ribbed shapes

(b) Precast deck module of loop joint with ribbed shapes

(c) Precast deck system of headed bar with ribbed shapes

(d) Precast deck system of loop joint with ribbed shapes

Fig. 3 Precast deck module and system

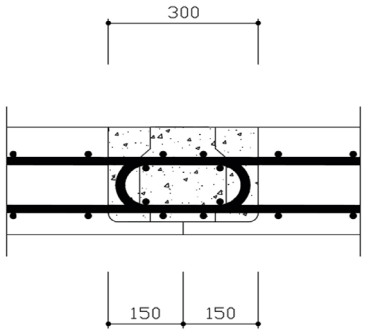


Fig. 4 Connection detail of LOOP300 B1, B2

하였고, 제안된 요철형 이음단면의 휨성능 평가를 위해 2차 요소실험을 수행하였다.

Fig. 4는 루프철근을 이용한 요철형 프리캐스트 바닥판 연결시스템의 이음부 연결상세를 나타낸 그림이다. 국내외 기준, 시방규정에 따라 후크 형태의 루프철근 겹이음 길이를 확보하면서 이음부의 몰탈타설부에 하부돌출콘크리트 단면을 적용하여 거푸집이 필요없어 급속시공이 가능하도록 하였다. 또한, 주철근 및 배력철근 배치를 고려하였으며, 사전 조립된 프리캐스트 바닥판과 이음부의 무수축몰탈 타설부의 부착강도 향상을 위해 이음부의 경계면에 요철형 단면을 적용하여 부착면적을 증대하였다.

4_실험 계획 및 내용

4.1 개요

제안된 요철형 프리캐스트 바닥판 연결기술의 휨성능 비교 검증을 위해 바닥판간 이음형식 (이음부 없는 일반RC 바닥판, 철근겹이음, 루프철근이음, 헤드철근이음 등), 이음부 폭 (200mm, 300mm), 철근간격 (등간격, 인접형), 이음부 단면형태 (직선형, 요철형)를 주요변수로 하여 Table 1과 같이 총 17개의 요소실험체를 제작하였다. 실험체의 프리캐스트 콘크리트의 설계기준강도는 40MPa, 이형철근의 항복강도는 400MPa, 바닥판간 이음부에 채워지는 무수축몰탈의 설계기준강도는 60MPa이다.

2개의 프리캐스트 바닥판 모듈간 이음부에 고강도의 무

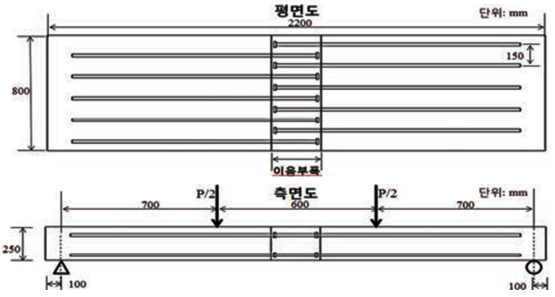


Fig. 4 Connection detail of LOOP300 B1, B2

Table 1 Connection details of specimens

실험체명		이음형식		이음부 폭 (mm)	철근간격 (mm)	이음부 단면
1차	RC1, RC2	이음부 없는 RC 바닥판		–	등간격	직선형
	loop 200	루프철근이음		200		
	loop 300			300		
	lap 200	일반겹이음		200		
	lap 300			300		
	lap 400			400		
	HB S1	헤드철근 이음	직선형	200	등간격	
	HB S2			200	인접형	
	HB C1		절곡형	200	등간격	
	HB C2			200	인접형	
2차	HB300 L1, L2	헤드철근 이음	직선형	300	인접형	
	HB300 B1, B2			300		요철형
	LOOP300 B1, B2	루프철근이음		300		

수축몰탈을 현장타설함으로써 Figs 3(c)와 3(d)와 같이 바닥판 모듈간 이음부가 일체화된다.

4.2 4점재하 휨실험

실험체의 제원은 폭 800mm, 두께 250mm, 길이 2200mm 동일하며, 주철근과 배력철근은 H19mm, 배근간격은 모두 150mm이다. 각 실험체의 연결상세는 다르지만 외형과 제원은 동일하며 순수휨이 재하되도록 양단 단순지지 조건에서 500kN 용량의 엑츄에이터를 이용하여 4점하중을 재하하였다. 실험체의 제원 및 하중재하방법은 Fig. 5와 같다.

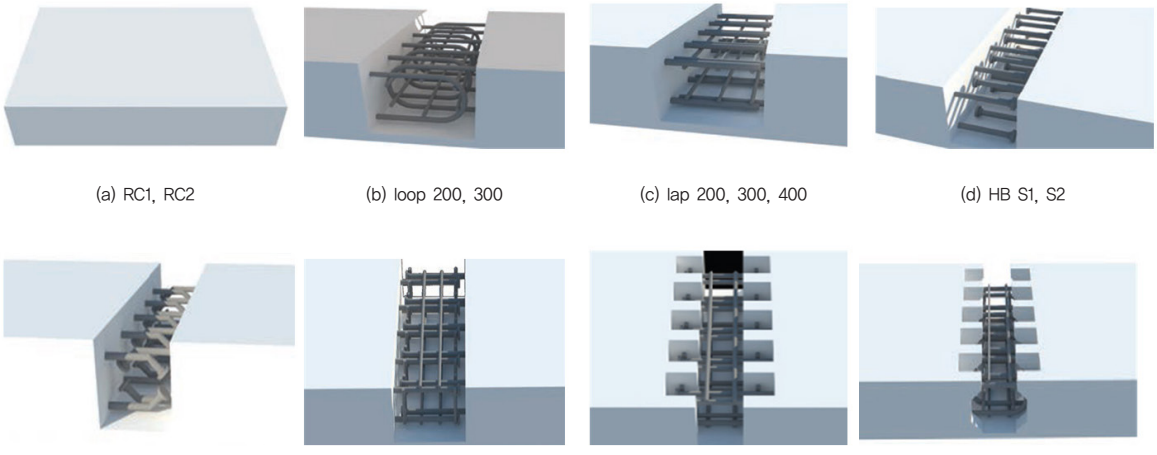
4.3 실험체 계획

프리캐스트 바닥판의 이음부 형식에 따른 휨성능 비교를 위해 Table 1과 같이 총 17개의 실험체를 제작하였다. 기준 실험체로서 이음부가 없이 일체로 제작된 RC바닥판 실험체(RC1, RC2) 2개를 제작하였고, 프리캐스트 바닥판 이음부를 모사하여 다음과 같이 15개의 요소시험체를 제작하였다. 루프철근 형식으로 이음부폭이 200mm, 300mm이며, 직선형 이음단면을 갖는 것 (loop 200, loop 300), 일반 겹이음 형식으로 이음부 폭이 200mm, 300mm, 400mm로 변화시킨 것(lap 200, lap 300, lap 400), 직선헤드철근 형식으로 이음부 폭이 200mm이고 철근간격을 등간격으로 배치한 것 (HB S1)과 직선헤드철근 형식으로 이음부 폭이 200mm이고, 철근간격을 부등간격 즉, 인접형으로 배치한 것 (HB S2), 절곡된 헤드철근 형식으로 이음부 폭이 200mm이고, 철근간격을 등간격을 배치한 것 (HB C1), 절곡된 헤드철근 형식으로 이음부 폭이 200mm이고, 철근간격을 부등간격 즉, 인접형으로 배치한 것 (HB C2), 직선 헤드철근 형식으로 이음부 폭이 300mm이고, 직선형 이음단면을 갖는 것 (HB300 L1, HB300 L2), 직선 헤드철근 형식으로 이음부 폭이 300mm로서 요철형 이음단면을 갖는 것 (HB300 B1, HB300 B2), 루프철근 형식으로서 이음부 폭이 300mm이고, 요철형 이음단면을 갖는 것 (LOOP300 B1, LOOP300 B2)이다.

Fig. 6은 각 실험체의 이음부 연결상세를 나타낸 그림이다.

4.4 측정항목 및 게이지 설치

각 실험체의 전체적인 게이지 측정위치는 Fig. 7과 같으며, 게이지의 측정항목 및 종류 등은 Table 2에 수록하였다. 이를 통해 하중재하에 따른 중앙부 처짐과 상부철근 및 하부철근 변형률, 콘크리트 변형률, 균열폭 등을 측정하였다.



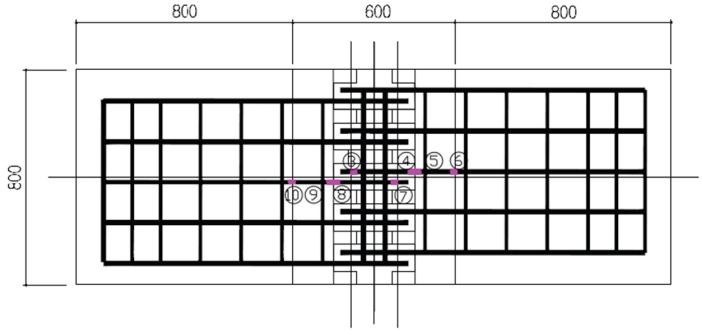
(a) RC1, RC2 (b) loop 200, 300 (c) lap 200, 300, 400 (d) HB S1, S2

(e) HB C1, C2 (f) HB300 L1, L2 (g) HB300 B1, B2 (h) LOOP300 B1, B2

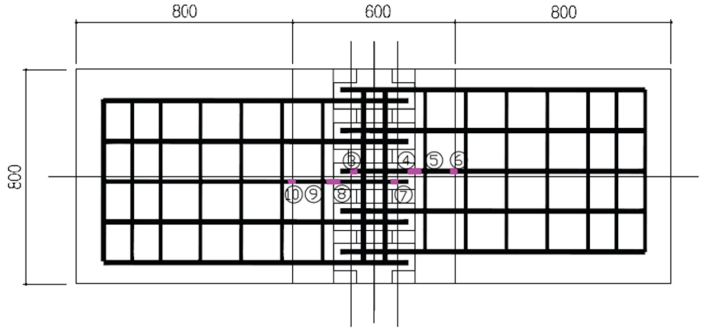
Fig. 6 Connection detais of specimens

Table 2 Gage configuration

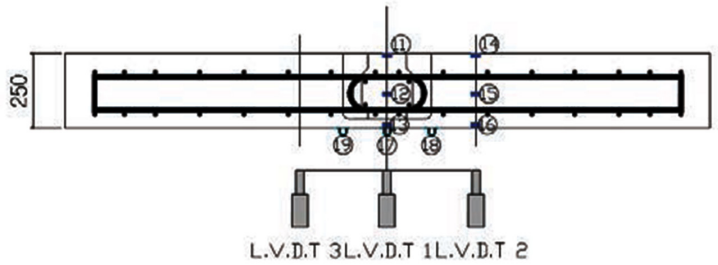
측정항목	게이지의 위치	사용 게이지	측정시기
처짐	지간중앙 및 하중재하점 : 3개소	L.V.D.T	하중재하시
철근 변형률	상부철근 : 2~4개소 하부처근 : 8~10개소	steel gage	하중재하시
콘크리트 변형률	지간중앙 상, 중, 하 : 3개소 하중재하점 상, 중, 하 : 3개소	concrete gage	하중재하시
균열	지간중앙 하부 및 균열발생부위 : 3개소	crack gage	균열발생시



(a) Strain gage of top reinforcements



(b) Strain gage of bottom reinforcements



(c) Concrete strain gage and crack gage

Fig. 7 Gage schematic of precast concrete deck

4.5 실험체 제작 순서

프리캐스트 바닥판의 실험체 제작 순서는 Fig. 8과 같다. 프리캐스트 바닥판의 거푸집을 제작하고 철근조립 후 프리캐스트 콘크리트를 타설한다. 그 후 두 개의 바닥판 모듈 사이의 이음부에 철근게이지를 설치 후 무수축몰탈을 타설하여 이음부를 일체화하였다.

프리캐스트 바닥판 이음부의 휨성능 검증을 위한 4점재하 정적하중 실험은 Fig. 9와 같다.

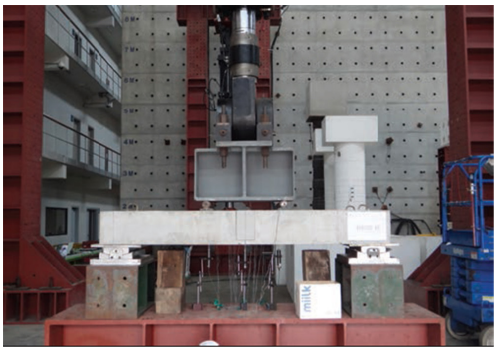
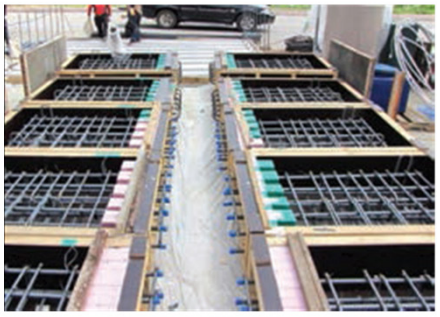


Fig. 9 View of flexural bending test



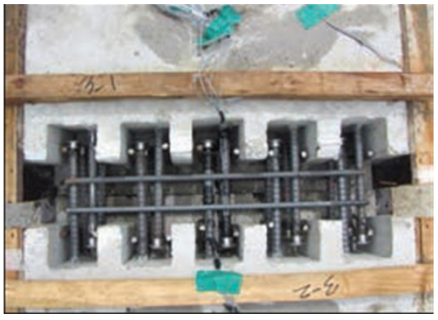
(a) Form making



(b) Rebar caging



(c) Concrete placement



(d) Strain gage installment



(e) Non-shrink mortar pouring



(f) Specimen test setup

Fig. 8 Production of specimens

5_실험결과 및 분석

5.1 하중-처짐 관계 분석

각 실험체의 하중재하에 따른 하부 인장철근의 항복하중, 최대하중, 파괴양상 등을 정리하면 Table 3과 같다.

Table 3. Test results summary

실험체명	이음형식	이음부 폭 (mm)	항복하중 (kN)	최대하중 (kN)	파괴양상	이음단면 형태
RC1, RC2 (이음부 없는 RC 바닥판)	일체형	없음	390	470	연성	직선형
loop 200	루프철근이음	200	280	350		
loop 300		300	280	400		
lap 200	일반겹이음	200	–	170	취성	
lap 300		300	–	260		
lap 400		400	–	320		
HB S1	헤드철근이음	200	–	170		
HB S2			–	200		
HB C1			–	110		
HB C2			–	140		
HB300 L1, L2	헤드철근이음	300	300	420		
HB300 B1, B2			300	455	연성	요철형
LOOP300 B1, B2	루프철근이음	300	300	470		

이음형식별로 실험체의 파괴시까지 하중을 재하하면서 중앙처짐을 관찰한 하중-처짐 곡선은 Fig. 10과 같다.

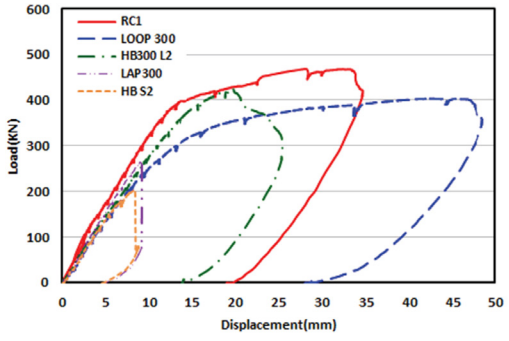


Fig. 10 Load-displacement curve (by types)

Fig. 11은 이음부 폭 300mm이고, 루프철근과 헤드철근 이음형식으로 직선형과 요철형 이음단면 형태별로 파괴시까지 하중재하에 따른 중앙처짐을 관찰한 하중-처짐 곡선이다.

Table 3과 Fig. 10으로부터 기준실험체인 이음부가 없는 일체형 RC바닥판 (RC1, RC2)의 경우 최대하중이 470kN로서 휨강도가 가장 크며 연성파괴 거동을 보임을 알 수 있다. 또한 일반겹이음 (lap 200, lap 300, lap 400)과 이음부 폭이 200mm 이하인 헤드철근 형식 (HB S1, HB S2, HB C1, HB C2)은 이음부의 소요성능을 발휘하기 전에 정착길이 미확보로 인하여 이음부 균열발생과 동시에 뽑힘에 의한 취성파괴가 발생하였다. 그리하여 이음부 폭 300mm 이상 확보가 휨강도의 증가에 중요한 역할을 하는 것으로 판단된다 (HB300 L2, HB300 B2).

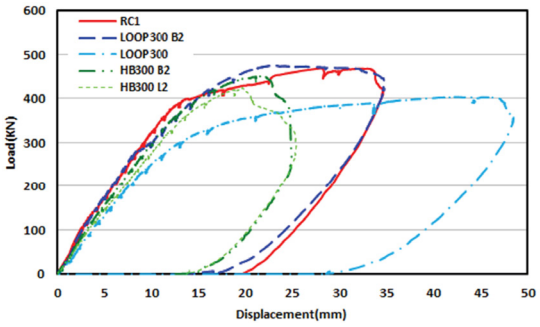


Fig. 11 Load-displacement curve (by connection shapes)

Table 3으로부터 이음부 폭 300mm이고 헤드철근 이음형식을 갖는 요철형 단면 (HB300 B1, B2)의 최대하중이

455kN으로서 직선형 단면 (HB300 L1, L2)의 420kN에 비해 10%내외 휨강도가 큼을 알 수 있다. 특히 Fig. 11로부터 루프철근 이음 형식의 이음부폭이 300mm이고 요철형 이음단면을 갖는 실험체 (LOOP300 B2)가 기준실험체 (RC1, RC2)와 거의 동등수준의 휨강도과 연성거동을 보여줌으로써 가장 양호한 이음부 연결상세임을 알 수 있다.

5.2 하중-변형률 관계 분석

Fig. 12는 하중재하에 따른 하부 인장주철근의 변형률을 기록한 하중-철근변형률 곡선이다. Fig. 12에서 요철형 이음단면의 루프철근 이음 실험체 (LOOP300 B2)가 기준실험체(RC2)와 거의 동등수준의 철근응력을 보여준다. 이음부 폭이 300mm로서 동일할 경우 루프철근 형식으로서 요철형 이음단면이 정착 및 이음 효과가 가장 양호하다고 판단되며, 철근 항복변형률 2000μ m 이후에 소성거동을 보이므로 철근의 정착 및 이음이 충분히 이루어져 있는 것으로 판단된다.

Fig. 13은 실험체별 하중재하에 따른 부재 중앙부 상단의 콘크리트 변형률을 기록한 하중-콘크리트변형률 곡선이다. 전체적으로 모든 실험체가 거의 유사한 압축거동을 보여준다. 하중 증가에 따라 콘크리트 상부의 휨압축으로 콘크리트 압축변형률이 증가하는데 Fig. 11의 하중-처짐 곡선과 유사한 거동을 보임을 알 수 있다.

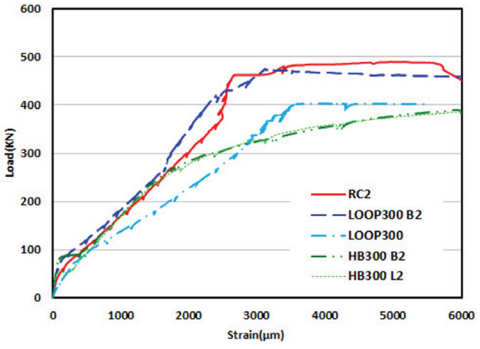


Fig. 12 Load-strain curve (bottom of reinforcment)

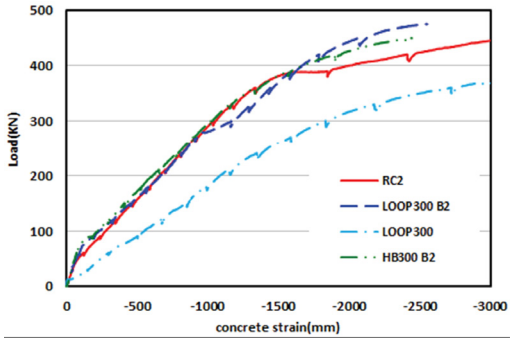


Fig. 13 Load-strain curve (top of concrete)

5.3 하중-균열 관계 분석

Fig. 14는 하중재하에 따른 이음부 하부의 균열폭을 측정한 곡선이다. 전체적으로 하중 증가에 따라 균열폭이 증가하는데 특히 사용성 설계시 중요한 균열폭 0.2mm일 때의 균열하중을 살펴보면 요철형 이음단면의 실험체 (LOOP300 B2, HB300 B2)는 176kN, 162kN이고 직선형 이음단면의 실험체 (LOOP300, HB300 L2)는 80kN, 115kN으로서 요철형이 직선형보다 균열하중이 커 보다 양호한 균열거동을 보여준다.

Fig. 15는 실험체별 파괴하중 재하시까지의 균열양상을 보여준다. 이음부 단면형태가 요철형인 경우 직선형인 경우와 비교하여 파괴시까지 균열양상이 기준실험체 (RC2)와 가장 유사한 거동을 보임을 알 수 있다. 이는 요철형상에 의하여 이음부에서의 부착강도 증가로 휨강도 및 균열저항성능이 높아져 초기균열을 이음부 위치가 아닌 전단면에 걸쳐 발생하도록 유도한 것으로 판단된다.

6_결 론

기존 프리캐스트 바닥판의 연결기술을 개선하고자 종방향 내부긴장재 없이 이음부 무수축몰탈 타설로 급속시공이 가능한 요철형 이음단면을 갖는 프리캐스트 교량 바닥

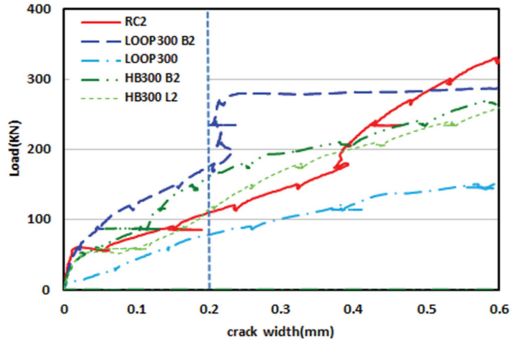


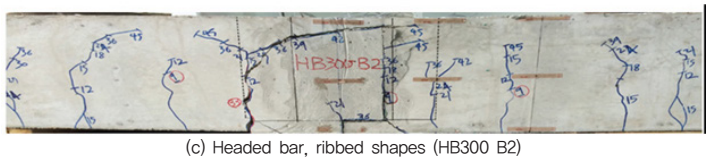
Fig. 13 Load-strain curve (top of concrete)



(a) RC deck without joint (RC2)



(b) Loop joint, ribbed shapes (LOOP300 B2)



(c) Headed bar, ribbed shapes (HB300 B2)



(d) Headed bar, straight shapes (HB300 L2)

Fig. 15 Cracks of specimens

판을 제안하였다. 휨성능 요소실험을 통해 제안된 프리캐스트 바닥판의 극한거동 및 균열양상을 살펴보고 일반 RC 바닥판의 거동과 비교하였다. 이로부터 얻은 결론은 다음과 같다.

1) 제안된 루프철근 이음형식의 요철형 이음단면을 갖는 프리캐스트 바닥판의 극한거동이 이음부가 없는 일체형 RC바닥판과 거의 동등수준의 휨성능과 연성거동을 발휘하여 가장 양호한 거동을 보이는 것으로 판단된다.
2) 실험체별 하중-처짐 곡선, 하중-변형률 곡선, 하중-균열폭 곡선, 하중재하시 균열양상 등의 실험 결과로부터 요철형 단면이 직선형 단면보다 우수한 휨강도와 균열거동을 보이는 것으로 판단된다. 이는 요철형상에 의하여 이음부의 부착면적 증가로 휨강도 및 균열저항성능이 높아지고 초기균열을 이음부가 아닌 전단면에 걸쳐 발생하도록 유도한 것으로 사료된다.
3) 향후 제안된 프리캐스트 바닥판에 대해 거더와의 합성거동, 부모멘트 구간의 균열제어, 차량하중에 의한 바닥판의 전단거동 등에 대한 연구를 수행할 예정이다.

감사의 글

본 연구는 국토교통부/국토교통과학기술진흥원 건설기술연구사업의 연구비지원 (13건설기술A01)에 의해 수행되었습니다.

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