การศึกษาเชิงทดลองและวิเคราะห์แผ่นพื้นหล่อสำเร็จแบบแบ่งส่วนและรอยต่อแบบห่วง

ชำนาญ ดวงจรัส[1*] และ สิทธิชัย แสงอาทิตย์[2]

บทคัดย่อ

บทความนี้นำเสนอผลการทดสอบและผลการวิเคราะห์แผ่นพื้นสำเร็จแบบแบ่งส่วนซึ่งประกอบด้วยแผ่นคอนกรีตหล่อสำเร็จพร้อมคานกว้างจำนวน 4 แผ่นประกอบต่อกันด้วยรอยต่อแบบห่วง ทำการทดสอบจำนวน 3 ชุดแต่ละชุดเป็นพื้นช่วงเดียวขนาด 3200x3200 มิลลิเมตร รองรับด้วยเสาเหล็กขนาด 200x200 มิลลิเมตร ที่มุมพื้น การทดสอบกระทำภายใต้น้ำหนักกระจายสม่ำเสมอโดยใช้ถุงทราย ทำการวิเคราะห์ระบบพื้นด้วยวิธีไฟไนต์เอลิเมนต์โดยสมมุติสมบัติของวัสดุแบบไม่เชิงเส้น จากผลการทดลองพบว่าแผ่นคอนกรีตมีพฤติกรรมเชิงเส้นถึง 40% ของกำลังสูงสุด หลังจากเกิดรอยร้าวแผ่นพื้นมีพฤติกรรมไม่เป็นเชิงเส้นและเหล็กเสริมหลักเกิดการครากเมื่อรับน้ำหนักประมาณ 65% ของน้ำหนักสูงสุดที่อัตราส่วนการแอ่นตัวสูงสุดบริเวณกลางช่วงพื้นต่อความยาวช่วงเสาประมาณ 0.003 แผ่นพื้นทั้งหมดมีความเหนียวสูงเมื่อเกิดพิบัติโดยมีแฟคเตอร์ความเหนียวประมาณ 6 และรูปแบบการพิบัติเป็นแบบก้าวหน้า เมื่อเปรียบเทียบผลการวิเคราะห์กับผลการทดลองสรุปได้ว่าแผ่นพื้นสำเร็จรูปแบบแบ่งเป็นส่วนสามารถสร้างให้แข็งแรงเช่นเดียวกับการสร้างแบบเดิมเพื่อรองรับการใช้งาน 3.0 ถึง 4.0 กิโลนิวตันต่อตารางเมตร เมื่อรอยต่อแบบห่วงเชื่อมต่อพื้นได้อย่างสมบูรณ์

คำสำคัญ: ไม่เชิงเส้น, หล่อสำเร็จ, แผ่นพื้น, ความลึกทั้งหมด, ไฟไนต์เอลิเมนต์

[1] นักศึกษาปริญญาเอก สาขาวิชาวิศวกรรมโยธา สำนักวิชาวิศวกรรมศาสตร์ มหาวิทยาลัยเทคโนโลยีสุรนารี

[2] รองศาสตราจารย์ สาขาวิชาวิศวกรรมโยธา สำนักวิชาวิศวกรรมศาสตร์ มหาวิทยาลัยเทคโนโลยีสุรนารี

[*] ผู้นิพนธ์ประสานงาน โทร. +669 7014 1960 อีเมล: chamnand@gmail.com

Experimental and Analytical Study of Segmental Precast Slabs with Loop Joints

Chamnan Duangjaras[1*] and Sittichai Seangatith[2]

Abstract

This paper presents the testing results and analytical results of segmental precast slabs, formed by four panels of segmental precast units with band beams and connected together with loop joints. Three sets of the single span of 3200x3200 mm slabs were tested. They were simply supported at each corner by steel columns of size 200x200 mm and were tested under uniformly distributed load steps by means of sand bags. The slabs were also analyzed by using the nonlinear finite element method with assuming nonlinear material properties. From the experiments, it was found that the slabs have a linear behavior up to 40% of its ultimate load. After cracks occurred, the slabs behave nonlinearly and yielding of the main steel reinforcement occurred at the load about 65% of its ultimate load at the ratio of the maximum deflections to span length about 0.003. Also, all slabs have very high ductility at their failure with the ductility factor about 6 and the mode of failure can be considered as a progressive failure. Comparing the analytical results with the experimental results, it can be concluded that the segmental precast slabs can be built as strong as the conventional slabs to sustain service loads for the ranges of 3.0 to 4.0 kN/m^2, providing that the loop joints connect the precast units perfectly.

Keywords: nonlinear, precast, slabs, full depth, finite element

[1] Doctoral student, School of Civil Engineering, Institute of Engineering, Suranaree University of Technology
[2] Associate Professor, School of Civil Engineering, Institute of Engineering, Suranaree University of Technology
[*] Corresponding Author, Tel. +669 7014 1960 e-mail: chamnand@gmail.com

1. Introduction

Conventional precast slabs are widely used in building construction due to better quality control in manufacturing and fast installation on site. Most of these slabs are partially pre-cast which concrete topping is required. Moreover, these precast slabs are one-way slabs requiring beams support at each slab end. Today, precast slabs are made in various section shapes such as solid planks, hollow cores, and double-T. Full depth precast slabs are mostly used as bridge decks to accelerate installation [1], [2] and are used as concrete pavement to reduce construction time for repairing of concrete highway [3]. To form a large panel, the full depth precast slabs are connected together with loop joints which are widely used connections for this slab type. The effectiveness of the joint was studied under static and fatigue loading by using concrete beams [4], [5] and it was found that the beam with the joint yields strength and stiffness similar to those of the conventional beams without joints provided that the joint width was sufficient for development length of the reinforcement. The joints were also tested under tension [6] and it was found that the failure of the joint influenced by various parameters such as the overlapping length, the spacing of the loops and the amount of the transverse reinforcement. These studies are implemented in pavement and bridge decks which require short time construction avoiding traffic interference. Comparing with the two-way slabs with the same design variables, the effectiveness of these slabs is lower due to the one-way distribution of the internal shear and moment in the slab. Therefore, to improve the effectiveness of the slabs, they should be segmentally and fully precast as two-way slabs with band beams. Also, to accelerate the construction, the slabs should be without topping. For installation, each slab is connected together with loop joints and supported directly by columns. The installed slabs are, hence, similar to flat slabs or slabs with band beams and in general this can be used as building floors. This paper presents the testing results and analytical results of segmental precast slabs, formed by four panels of full depth precast units with band beams connected together with loop joints. The slabs were simply supported and tested under uniformly distributed load by means of sand bags.

2. Experimental works

2.1 Details of segmental precast slabs

Three sets of precast units were cast and each unit consisted of a slab portion and partial beam portions. Thickness of slab portions were 80 mm, 100 mm and 100 mm for the first set, the second set and the third set respectively. Based on standard concrete design theory, the slab was reinforced with two layers of rebars with diameter of 6 mm. Rectangular loops protruded alternately from this reinforcement as shown in Fig. 1. The horizontal length of each loop was based on the development length in according to the ACI code which is 130 mm length for the bar of diameter 6 mm in concrete strength of 30 MPa. Beam portions were reinforced with two rebars with diameter of 9 mm at top and bottom layers and tie with the stirrup with diameter of 6 mm at spacing of 150 mm. Dimensions and reinforcement of each precast unit are shown in Fig. 1 and Table 1. The single span slab was formed by four single units connected at mid-span as shown in Fig. 2. These connection strips were poured with in-situ concrete. Reinforcement details and dimensions are shown in Table 1.

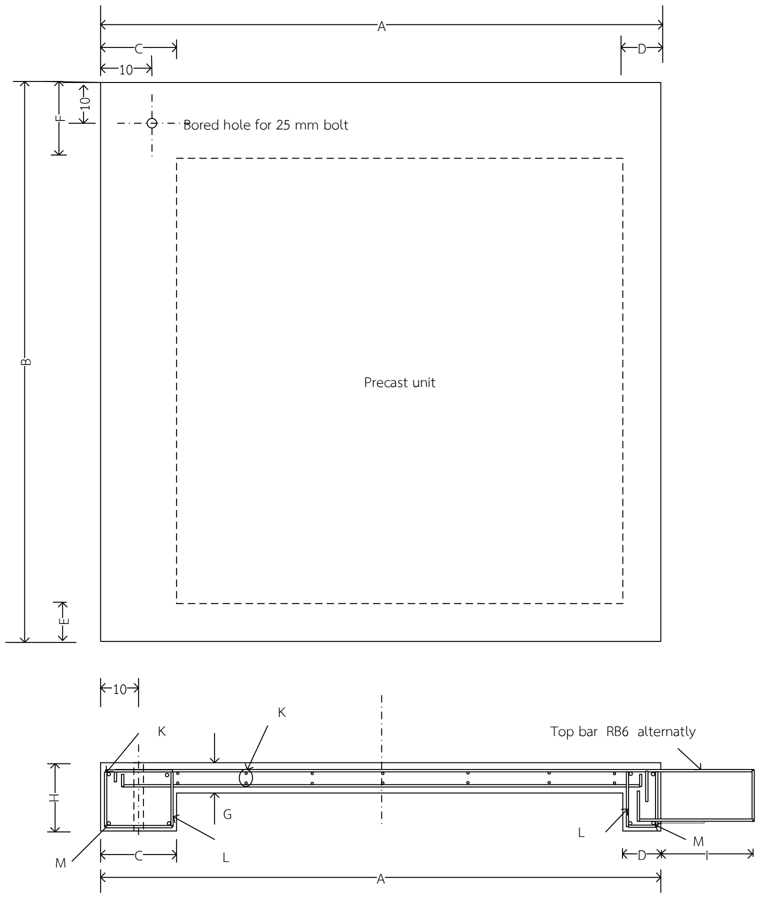


Figure 1 Dimensions and reinforcement of a precast unit

Table 1 Dimensions and reinforcement of precast units

set	Precast unit dimensions(mm)										Reinforcement		
	A	B	C	D	E	F	G	H	I	J	K	L	M
1	1475	1475	200	200	100	100	80	180	240	250	7RB6@195#	RB6@150	4RB9
2	1500	1500	200	200	100	100	100	160	290	200	7RB6@200#	RB6@150	4RB9
3	1450	1450	200	200	100	100	100	200	290	300	7RB6@230#	RB6@150	4RB9

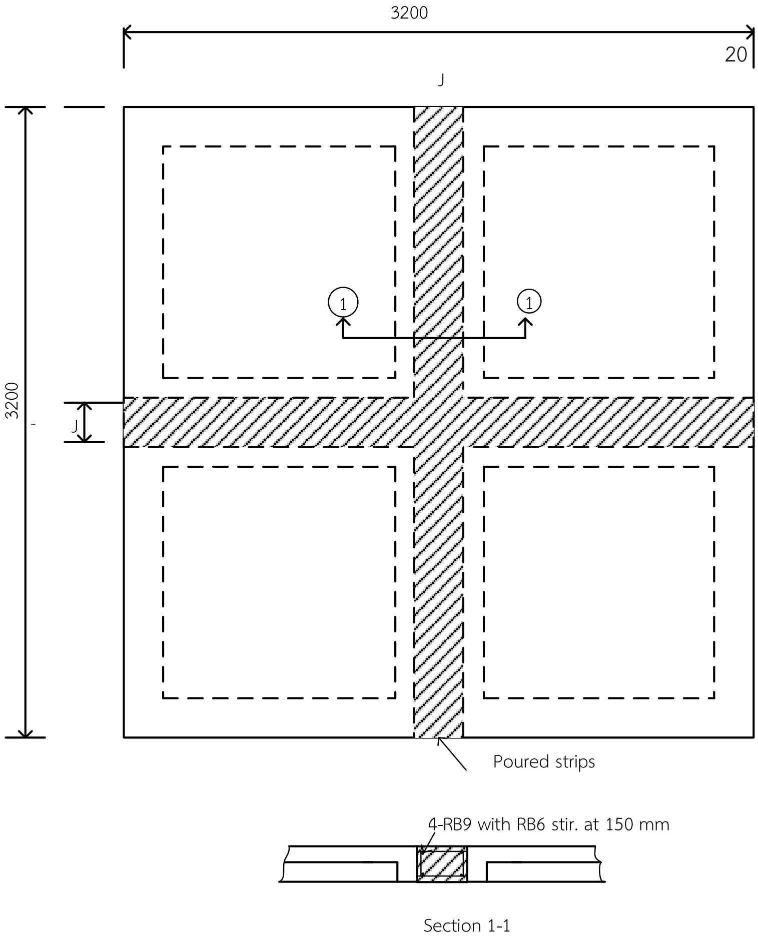


Figure 2 Dimensions and reinforcement of a single span slab with Loop joint connections

2.2 Material properties

Concrete of all precast units was ready-mixed concrete. Five concrete samples were cast and cured in water for each concrete batch and curing time was at least 28 days. The concrete for poured strip was cast in-situ. All samples were tested in according with ASTM C39. The averaged compressive strength are shown in Table 2. The reinforcement used was round bars. Five samples of each bar size were tested in according with ASTM A370. The averaged tensile strength of each bar size is shown in Table 2.

Table 2 Material properties

Cylinder f'_c (MPa)		Rebars (MPa)			
Precast unit	Poured strip	RB6		RB9	
		f_y	f_u	f_y	f_u
37	40	270	347	297	410

2.3 Test set-up

The slab was simply supported at each corner by the stiff steel column of size 200x200 mm forming the single span of 3000x3000 mm center to center of columns. Deflections were measured at the center of each precast unit and at mid-span using Kyowa DT-A100 LVDT.

Elongation in bottom reinforcement of the mid-span beam was measured by using strain gauges attached to the steel bars at mid-span position. The deflection data and strain data was automatically collected by Yokokawa DA100 data logger. Between the contacted surfaces of the slab and the column sections were filled with white cement mortar for the purpose of leveling. All surfaces of the slab were painted with the mixture of white cement and water to facilitate detection of concrete cracking. Sand bags were used to apply uniformly distributed load to the slab. The overall set-up is shown in Fig. 3.

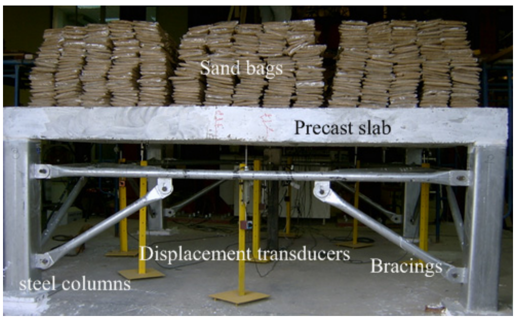


Figure 3 Schematic diagrams of general set-up

2.4 Testing

The surface area of the slab was divided into grid of 3x3 which each grid covers area of 1.0 m^2. Total area of the slab surface was 9.0 m^2. Then the slab was loaded step by step using sand bags uniformly distributed over each grid area. Each load step was about 300 N/m^2. A preload of approximately 900 N/m^2 was applied to seat the testing slab before the beginning of each test. At each load step, the total weight and the deflections and also stains were recorded after load sustaining for about 3 minutes and the slab was observed thoroughly for concrete cracking. Then the next load step was begun. The slab was loaded to the point where the deflections increased dramatically with little or no increase in load or severe cracks occurred.

3. Test results

3.1 Load-deflection curves

Fig. 4 shows the uniformly distributed load versus mid-span deflection curves. It was found that slab sets have a linear behavior up to 2.5 kN/m^2 . After cracks occurred, the slab behaved nonlinearly and yielding of the main steel reinforcement occurred at the load about 4.0 kN/m^2 with the center mid-span deflection of about 12 mm. It is equivalent to the span-to-deflection ratio of about 250. After yielding, the slab stiffness reduced progressively with the increasing of the load, causing a significant increase of the mid-span deflections for each load step. Maximum load of slab set 1 was about 6.0 kN/m^2 with the maximum mid-span deflection of about 43 mm. The maximum load of slab set 2 and slab set 3 were virtually the same for about 6.30 kN/m^2. The ratio of the maximum displacement to that at yielding is larger than 4.

3.2 Concrete cracking and deformation

In this study, the surfaces of the precast concrete in contact with the poured concrete were smooth surfaces which are to simulate the worst case of connection between the new and old concrete. Fig.5(a), Fig. 5(b) and Fig. 5(c) show that the cracks occurred at the joint in the vicinities of the interfaces of the precast unit and the poured concrete at the mid-span at various load steps.

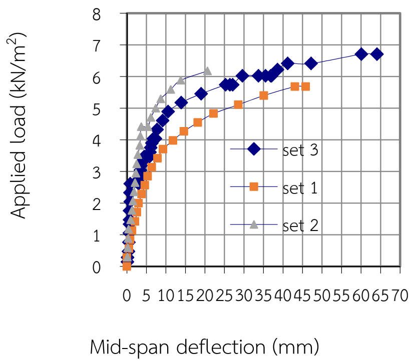
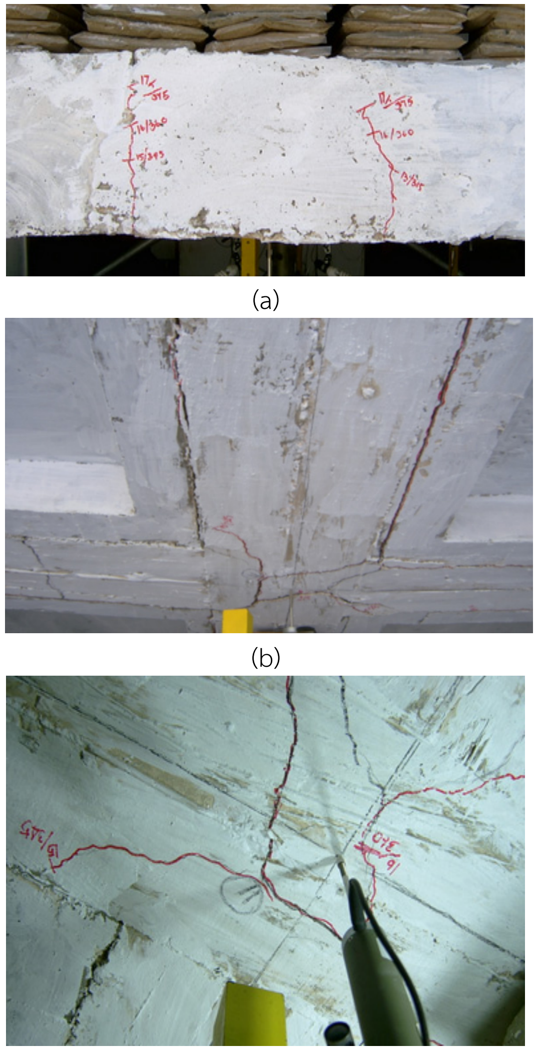


Figure 4 Load-mid span deflection curves of all slabs

It is noted that reinforcement at the sections of these interfaces was less than that in slab portions or that in the sections between the interface surfaces, causing the interface section to be the weakest sections in the slab system. Hence, the concrete cracking is easily occurred along the interface lines. These cracks also show that there was significant tensile force in the transverse direction of the beams. Conversely, the interfaces can be reinforced so that the section capacity are equal to or even stronger than that of the slab potions or the beam sections. These cracks were occurred throughout the slab width. At the center of the mid-span of the slab, the complex cracks were observed as shown in Fig. 5(c). In addition, there were no cracks found in other parts of the slab. The maximum deflection at mid span of set 3 was about 63 mm which is equivalent to the span-to-deflection ratio of about 50, causing the slab end at each column to be tilted upward. This indicates that the slab has very high ductility at failure.



(a)

(b)

(c)

Figure 5 Concrete cracking at the joint interfaces (a), (b) and (c) at bottom at mid-span

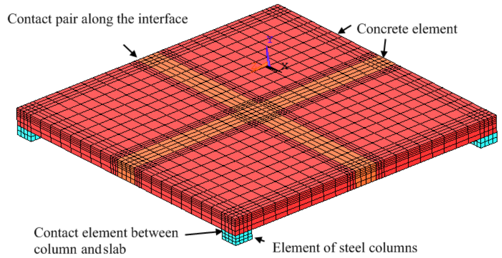
4. Analytical works

4.1 Introduction

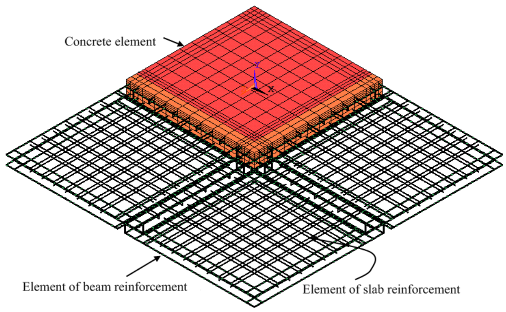
In the study of slab behavior under various conditions such as geometry, loading, composite action etc., experimental works can directly reveal the behavior of the slab. However, since the experimental works are expensive and give only some parameters that influence on slab behavior. Numerical simulations of slab behaviors are then necessary. In the past few years, a number of researchers have used the finite element that includes the bond slip, dowel action and tension stiffening to analyze this kind of problems [7]. It was found that the finite element yields results that are fairly in good agreement with experimental results. Also, a three-dimensional finite element model was used to simulate bridge deck behavior in Virginia, USA using ABACUS. The accuracy of the model was verified with hand calculation and the validated model was used to evaluate the Route 621 Bridge over the Willis River [8]. The material model in nonlinear analysis plays an important role for accuracy of prediction. An elastic strain hardening, plastic stress-strain relationship with a non-associated flow rule were used to model concrete in compression and an elastic brittle fracture was assumed for concrete in tension. The steel was modeled by idealized bilinear curves identical in tension and compression. The models yielded good agreement with experimental data [9]. A nonlinear finite element code was formulated to take into account the high strength concrete of slabs which were modeled with 20-node isoparametric brick element. The material model for cracked concrete was incorporated in the analysis and analysis results were in good agreement with test data [10]. In this study, the segmental precast slabs have some features that are different from the conventional slabs such that portions of slabs were precast and connected together in installation. This creates connection lines where new and old concrete to be in contact. Moreover, transferring of load across the connection by means of loop bars or splice bars together with in-situ concrete are distinct characteristics of this slab system. Hence, the finite element model for this slab system should account for all features of the slabs. The ANSYS [13] has features that can be used to model this slab system. In this modeling, the concrete of the slab and beams were modeled by the SOLID65 element which is a 8-node brick element without reinforcing bars having three degree of freedom at each node; translation in x , y and z directions. This element can be simulated cracking, crushing and plastic deformation of concrete. All support columns are assumed in linear elastic behavior and modeled with the SOLID45 element which is also a 8-node brick element similar to the SOLID65. All reinforcement was discretely modeled by the LINK8 element which is a spa element having two nodes and three degree of freedoms at each node; translation in x, y and z directions. The element can be simulated-plasticity, large deformation and stress stiffening of the reinforcement. Interface surfaces between new and old concrete of the precast unit and the in-situ concrete in connection lines were modeled by contact pair elements; TARGE170 and CONTA174. These element pairs can simulate sliding, and separation of the contact surfaces. Also at all support columns at each corner of the slab where column sections contacted to the bottom of the slab, the contact was also modeled by this contact pairs.

4.2 Meshes of finite elements

The slab was discretized into thin layers along the thickness and uniform sizes throughout the slab volume. Also the integrated beams and the support columns were discretized in the same manner. The reinforcement was discretely modeled by assuming that perfect bond occurs between the concrete and the reinforcement. Hence the Link8 elements were connected to the SOLID65 elements at each node at bar level in concrete. The results of discretization of concrete and reinforcement are shown in Fig. 6.



(a)



(b)

Figure 6 Meshes of concrete and steel columns (a) and steel bars(b)

4.3 Material models

4.3.1 Material model of concrete

There are various concrete models available in literatures but the expression of Maekawa and Okamura [11] as shown in the equation (1) is adopted as a nonlinear material model of concrete. Since it matches with concrete characteristics in this study. This expression associated with the averaged compressive strength of the slab concrete yields the stress-strain curves as shown in Fig. 7.

$$\sigma = K_o E_o(\varepsilon - \varepsilon_p) \quad (1)$$

$$K_o = e^{-0.73x(1-e^{-1.25x})} \quad (2)$$

$$E_o = \frac{2.0 f_c'}{e_{peak}} \quad (3)$$

$$\varepsilon_p = \varepsilon_{peak} \langle x - \frac{20}{7}(1 - e^{-1.35x}) \rangle \quad (4)$$

$$x = \frac{\varepsilon}{\varepsilon_{peak}} \quad (5)$$

where

K_o = fracture parameter represents the damage of concrete

E_o = initial stiffness of concrete, MPa

ε_p = plastic strain corresponds the total strain

ε_{peak} = peak strain of concrete under compression

ε = strain

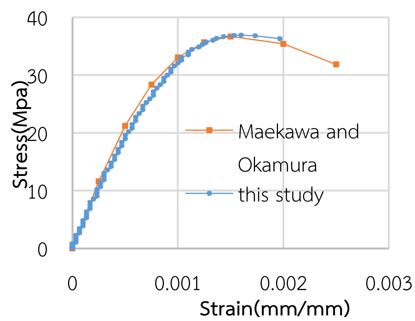


Figure 7 Typical stress-strain curves of concrete

4.3.2 Material model of steel

From the testing results of the steel, it was found that their behaviors are in accordance with Okamura's model [12] as shown in the equation 6. The stress is linearly elastic up to yielding point and after a certain yielding plateau, its behavior starts to be strain hardening in an exponential form. The 9 mm rebar yields the stress-strain curve as shown in Fig. 8. According to the experimental design, the steel columns behave linearly under the applied load.

$$\sigma = f_y + \left(1 - e^{\frac{(\varepsilon_{sh}-\varepsilon)}{k}}\right)(1.01f_u - f_y) : \varepsilon > \varepsilon_{sh} \quad (6)$$

$$k = 0.047\left(\frac{4000}{f_y}\right)^{2/3} \quad (7)$$

where

E_s = modulus of elasticity, MPa

f_y = tensile yield strength, MPa

f_u = ultimate tensile strength, MPa

ε = steel strain

ε_y = tensile yield strain

ε_{sh} = maximum tensile strain before strain hardening

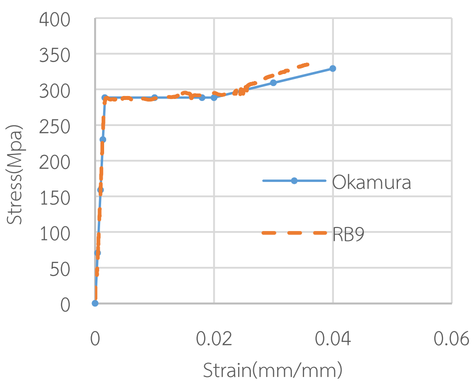


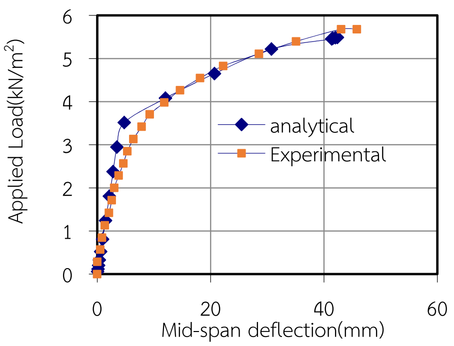
Figure 8 Typical stress-strain curves of RB9 steel bar

4.3.3 Contact pairs

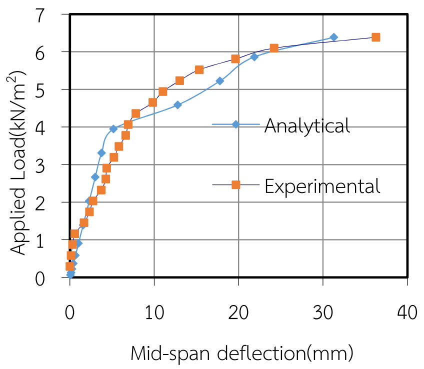
For each contact pairs, the material with higher strength was assumed to be the target surface while that with lower strength was assumed to be the contact surface. There are various contact types in ANSYS and, for this study, the interfaces between the precast units and the joint concrete could slide and separate under loading since the contact surface is smooth with low bonding. Hence the normal type of contact was employed. The friction between the contact surfaces could play an important role in the slab performance under loading. So In this study, ranges of friction coefficient (μ) used is between 0.1- 0.3 lower than 0.6 which is in the provision of the ACI code.

4.3.4 Analytical and experimental results

All of the material data and loading, including the finite meshes, were input to the program. Single load step with the smallest load step size of 0.001 and the maximum load step size of 0.1 were set for the nonlinearity. Convergence of the nonlinearity was controlled by using displacement L2 norm with the tolerance of 0.001. The analytical results and the experimental results are shown in Fig. 9(a), Fig. 9(b) and Fig. 9(c) for slab set 1, slab set 2 and slab set 3 respectively. These results show that the experimental results and the analytical results were in good agreement in the initial loading up to about 25 kN/m^2 when the slab had the linear behavior. The analytical results associated with the friction coefficient of 0.1 between the contact surfaces show in good agreement with experimental results after yielding. It implies that low bonding occurred in the interface between the old and new concrete. It also should be noted that cracking of concrete was not included in the analysis due to severely cracks occurred in bottom resulting in un-converged of the model. So results from the analysis based on nonlinearity of the material only. In the model, it is assumed that all parts of the slabs were perfectly bonded and without cracking of concrete so it is stiffer than the specimen that cracks can occur causing the discrepancy between the analytical results and the experimental results



(a)



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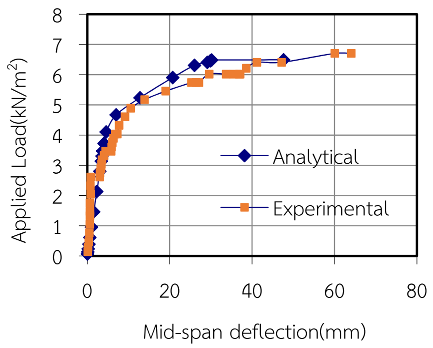

(c)

Figure 9 Analytical and experimental results of slab set 1(a), slab set 2(b) and slab set 3(c)

after concrete cracking before yielding. However, this discrepancy is small.The yielding load and the ultimate load from analytical results are in good agreement with the experimental results as shown in Fig. 9(a) to Fig. 9(c). Both of the analytical results and experimental results show high ductility of greater than 5.

5. Conclusions

From the study, it can be concluded that

1) The slab had a linear behavior up to 40% of its ultimate load. After cracks occurred, the slab behaved nonlinearly and yielding of the main steel reinforcement occurred at the load about 65% of its ultimate load with the span to-center mid-span deflection ratio of about 250. Also, the slab has very high ductility at the failure.

2) Low concrete bonding occurred between the interface of new and old concrete resulting on low stiffness of the slab and leading to lower slab capacity.

3) The finite element models used in this study give the predicted results that in good agreement with those from the experiments providing that low bonding is assumed at the interface between the new and old concrete.

4) Comparing the analytical results with the experimental results implies that the full depth precast slab could be built as strong as the conventional slabs to sustain service loads for the ranges of 3.0-4.0 kN/m^2 provided that loop joints are used to connect the precast units. Increasing friction between the precast units and the joint concrete could increase load sustaining capacity of the slab. The slabs performed in ductile manner with ductility factor greater than 5.

6. Acknowledgements

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