International Journal of Engineering & Technology, 7 (3.25) (2018) 458-462

SPC

International Journal of Engineering & Technology

Website: www.sciencepubco.com/index.php/IJET

Research paper

Analysis of Deflection and Moment Capacity on the Precast Beam of the Nk-Spircon Connecting System

Miftahul Huda [1], Hary Mursidi[2], Sri Wiwoho Mudjanarko[3*], Wahyu Mulyo Utomo[4]

[1]University of Wijaya Kusuma Surabaya, Surabaya. Indonesia
[2]Senior Engineer of Division IV, PT. Nindya Karya (Persero), Surabaya, Indonesia
[3]Narotama University, Surabaya. Indonesia
[3]Narotama University, Surabaya, Indonesia
**Corresponding author E-mail:sri.wiwoho@narotama.ac.id*

Abstract

This study is a case study on the planning of a multilevel building of the Faculty of Medicine, University of Wijaya Kusuma Surabaya, which aims to compare the strength of precast beam structure using NK-SpirCON connection system with conventional beam (cast in situ). The model of building structure and beam used is the result of consultant planning modified into precast beam with dimension and number of reinforcement according to planning result where connection between beam and column is used NK-SpirCON connection system. The test is done in collaboration with the number of specimens according to the number of variations of the existing beam type. The test is performed by gradually loading the load on the specimen to calculate the maximum load, maximum deflection and acceptable capacity moments of the beam. The results conclude that the precast beam using NK-SpirCON connection system has the ability to receive larger deflection and moment capacity than the conventional beam (cast in situ) with 29,81 % and 34,64 % difference. For further research it is advisable to research the shear strength of the beam and the strength of the connection between the beams and the columns due to various loads including quasi dynamic loads according to the prevailing regulations and regulations in Indonesia.

Keywords: *precast beam, NK-SpirCON connection, deflection, moment capacity.*

1. Introduction

In Indonesia, construction will be more developed if the government involves the private sector. Private facilities and infrastructure can be utilized in the construction of construsions [14].Construction industry is increasingly excited by the presence of precast concrete products that can be installed quickly and of excellent quality. The precast concrete is one of the alternative methods in building construction other than the conventional method generally used [1]. The precast method has several advantages and makes the construction process more effective and efficient [2]. Not only from the strength and stiffness of the structure, but also from the beauty of the architecture. Therefore, advanced-oriented structure planners will surely consider alternatives to the use of precast products for their design buildings [3]. With the use of precast then all the components that should be worked on the building and difficult to reach by supervisors to be supervised become easy to do under (in the factory) so that architects freely supervise the quality of products to be installed [4]. Because the production process is far from environmental pollution, concrete precast is said to be environmentally friendly concrete [5].

The precast concrete product, which is very important is the technology it uses. Not only the planning should be good but also need good implementation. Precast for finishing, which is displayed for beauty is more difficult than precast products that are just for structural components only [6]. Things to consider, for example: weather resistance, rainwater leakage, high precision, as well as the true details of the targets made so that the water effects of years do not leave any visible trace from the outside, as well as the details of the connection with its main building, how to anticipate the deformation of buildings that arise when there is an earthquake without experiencing performance degradation and others [1].

In addition, one of the causes of the increase in development costs is the waste of labor in the traditional implementation process. This tendency can only be minimized through the industrialization process by means of the use of fabrication results. Answering to these needs, precast concrete construction has grown rapidly and continues to grow in use [6]. The process of industrialization is achieved by the existence of repeated mass production and standard size such as column structure, beams, floors, top elements, wall panels and others. The component elements of the structure are produced in the field under fabrication conditions. In large parties, often places of precast fabrication are built in or adjacent to major construction sites [7].

Besides the advantages possessed by the precast system, there are also constraints of this system, namely the precast connection system [8]. The connection component of precast system is economically an expensive component, technically a part that must be proven by its behavior. The precast connection structure system generates criteria that vary in terms of structural behavior, which is highly dependent on the details and forms of precast structures to be applied in a particular construction [9][10]. Judging from technical factors, a precast system must have the same structural reliability criteria as conventional structural system, which is monolithic between the competence of each other and able to show good structural behavior in terms of pattern of cracking, ductility, energy that can be absorbed and transmitted and others [11][12].

In connection with the above description, at this time is often held modification of multi-storey building design model from the model of conventional concrete structure (cast insitu) into precast concrete structure. Therefore it is necessary to examine in depth the extent to which the structural strength difference to the static monotonic load generated from both models so that the pre planner can get a picture related to the modification. The main issues raised in this study are the extent to which strength and reliability of modified precast concrete beam structure using NK-SpirCON connection system compared with conventional (conventional) concrete structures and the extent of the beam power differences using NK-SpirCON connection system compared with the conventional system (cast in situ) in terms of deflection and moment of capacity capable of bearing to achieve the ductility level required by SNI. This study is limited to testing the strength of precast beam structures on aspects of the aspect: maximum strain and deflections and forces within the maximum acceptable structure of conventional plates and beams (before planning modifications).

2. Literature Study

2.1. Model Structure

The structural model of the building to be studied is idealized as in Figures 1 and Figure 2. The beam structures to be analyzed includes beams B1, B2, B3, B4 and B5. The division is based on the magnitude of each span of the beam and its location (at the edge or center span).

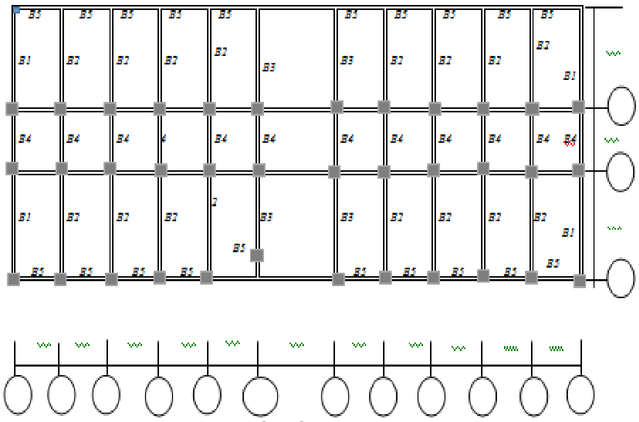


Fig 1:. The Floor Beam System

3. Methodology

3.1. Specification of Precast Structure Components

The design of Nindya NK-SpirCON precast structure system to be researched has the following specifications: (1) Quality of concrete fc‘ minimum = 35 MPa, (2) Reinforcing Steel BJTD or fy = 240 MPa (D < 13 mm), BJTD or fy = 390 MPa (D ≥ 13 mm) (3) System structure is open-shaped, (4). The precast components include: precast beam, preprinted column, beam-column assembly and precast platen, (5) The location of the connection lies in the beam component junction and the beam-columns connection junction. The connection device uses a NK-SpirCON connector.

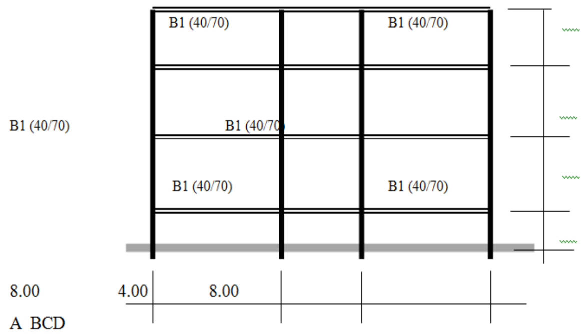


Fig 2:. Frame Cross Section (As: 1 to 12)

Explanation :

B1 = cross-sectional beam	L = 8m
B2 = cross middle beam	L = 8m
B3 = transverse middle beam	L = 8m (Frame as 6 & 7)
B4 = edge beam (center of cross)	L = 4m
B5 = edge beam (middle length)	L = 6m

Table 1: Dimensional Beams and Number of Reinforcement

Beam Type	Dimension (cm)	Number of Reinforcement		Shear Reinforcement Diameter
		Edge Beam	Middle Beam	
B1	40x60	5 D 19	5 D 19	d 10 – 10
B2	40x60	6 D 19	6 D 19	d 10 – 10
B3	40x70	8 D 19	8 D 19	d 10 – 10
B4	30x50	5 D 19	5 D 19	d 10 – 10
B5	40x60	7 D 19	7 D 19	d 10 – 10

Table 2: Moment Capacity and Maximum Deflection (Initial Design)

Beam Type	Dimension (cm)	Initial Design Results	
		M, maximum	δ, maximum
B1	40x60	287.24	68.02
B2	40x60	390.56	72.08
B3	40x70	360.12	69.99
B4	30x50	46.550	72.19
B5	40x60	150.28	65.12

3.2. Building Structure Data

Building plan and type of transverse and longitudinal beam can be seen in Figure 1. While the cross section of the building is idealized in Figure 2. The beam variation is differentiated into BM-1, BM-2, BM-3 and BM-4. Based on the initial planning results, the maximum deflection and moment that can be borne by several types of beams can be seen in Table 1 and Table 2 above.

3.3. Nindya-SpirCON Connection System

The connection system used in the precast structural system of NK-SpirCON is a wet connection where the connection between the reinforcement in the connection area between the precast components is used in the spiral form placed at the second meeting of the reinforcement. This form of connection is intended to be able to increase the adhesiveness of the reinforcement by a rebar confinement method so as to distribute loads that occur as uncoupled reinforcement forces. The NK-SpirCON material is a spiral shape that is wound around the intercrete encounter with the following specifications :

1) Minimum steel reinforcement: BJTP fy = 240 MPa
2) Diameter of reinforcing steel: db = 5 mm todb = 8 mm
3) Spiral length (I) : I = 100 mm to I = 300 mm
4) Inner spiral diameter: di = 20 mm to d_i = 50 mm

NK-SpirCON shape and dimensions as well as some alternatives for reinforcement applications can be seen in Figure 3 and Figure 4 above.

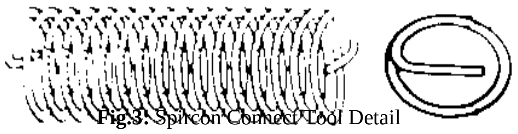
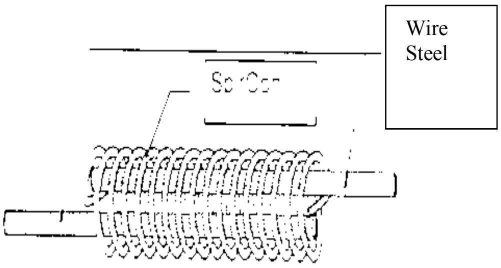


Fig.3: Spircon Connect Tool Detail



3.4. Material Quality Testing

This test is intended to obtain supporting data relating to the quality of materials used for the manufacture of specimens, which include concrete and steel reinforcement as well as strength test of reinforcement steel system using NK-*Spir*CON system in concrete

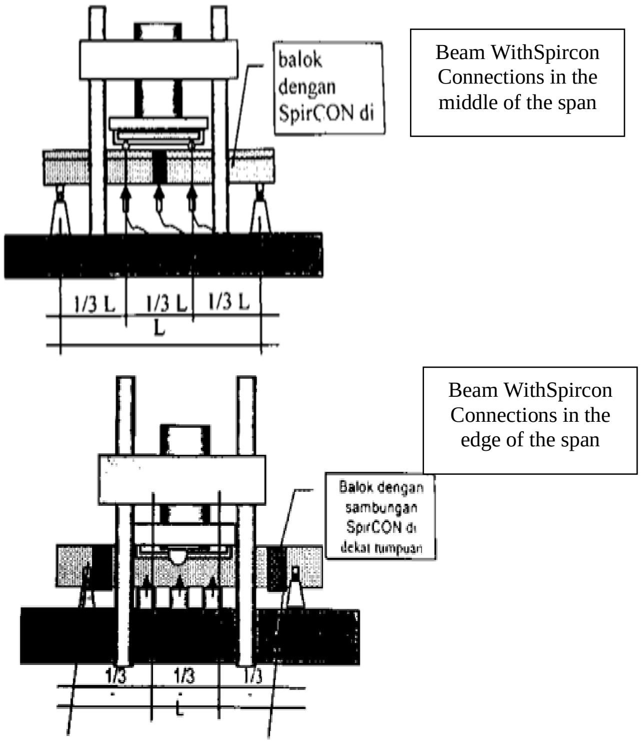


Fig 5: Set-Up Bending Beam Test

3.5. Strength Concrete Pressure Testing

The results of concrete strength test at various age and concrete quality evaluation result based on compressive strength of concrete cylinder test object can be seen in Table 3.

Tabel 3: Concrete Compressive Strength Test Result of Various Age

Ages (days)	Average compressive strength (Mpa)
3	25.96
7	30.47
14	35.93
28	39.20

3.6. Quality of Reinforced Steel

The reinforcing steel used in reinforcing balaok and precast columns and beam-koom connections comprises threaded reinforcing steel and plain steel bars. Strong Tensile Strength Test Results are listed in Table 4. And test result of tensile strength of steel, obtained load value causing melting and breaking reinforcement. And the value can be calculated the value of the melting stress (fy) and the breaking voltage (fc') of each diameter of the reinforcing steel used. The result of the calculation of the melting steel strain based on the equation is shown in Table 5.

$$\varepsilon_y = \frac{f_y}{E_s} \text{------------------------------} \quad (5.1)$$

Where: εy = strain of steel, fy = strength of steel (MPa), Es = modulus elasticity of steel (MPa). By taking the value modulus elasticity of steel of 200,000 MPa, it can be determined the value of the reinforcement strain stress as follows:

Table 4: Stress Test Result of Tension Reinforced Steel

Reinforcement	Ø (mm)	As (mm^2)	Load (N)		Stress (Mpa)	
			Yield	Fracture	Yield	Fracture
BJTD Ø 19	18.63	272.46	112000	156000	411.08	572.57
BJTD Ø 19	18.63	272.46	112400	157200	412.54	576.98
BJTD Ø 19	18.63	272.46	110000	154000	403.74	565.23
	18.63	272.46	106800	151200	391.99	554.95
	18.63	272.46	108000	151200	396.40	554.95
	18.63	272.46	112000	151200	411.08	554.95
				Average	404.47	563.27
				Standard Deviation (SD)	8.65	9.86
				Coef Variation (%)	2.14	1.75
				Fy	390.63	547.5
BJTD Ø 16	15.56	190.06	68000	108600	357.78	571.4
BJTD Ø 16	15.52	189.08	69000	109000	364.92	576.47
BJTD Ø 16	15.51	188.84	68000	109000	360.93	577.21
				Average	360.93	575.03
				Standard Deviation (SD)	3.64	3.16
				Coef Variation (%)	1.01	0.55
				Fy	355.11	569.97
BJTD Ø 16	6.10	29.21	13000	18200	445.05	623.07
BJTD Ø 16	6.10	29.21	13700	18500	469.02	633.34
BJTD Ø 16	6.10	29.21	13400	18500	458.75	633.34
				Average	457.61	629.92
				Standard Deviation (SD)	12.02	5.93
				Coef Variation (%)	2.63	0.94
				Fy	425.82	620.43

Table 5: Results of Calculation of Yield Steel Strain

Steel Reinforcement	f_{y-mean} (MPa)	ε_y
BJTD Ø 19	390.63	0.00195
BJTD Ø 16	355.11	0.00178
BJTD Ø 8	425.82	0.00219

This value of yield steel strain is used to determine the value of the yield load (Py) and the displacement when the yield reinforcement (Δy).

3.6. Deflection and Maximum Moment

Based on the test results conducted in the laboratory in accordance with the model and data beams that have been described above, the maximum load concentration test results and maximum deflections that can be borne by each type of beam obtained results as Table 6. The results of laboratory testing of B1, B2, B3 and B4 beam with centralized load in the middle of the span, resulted in the maximum test curve (P) and deflection (δ) relationship as shown in Table 6 to Table 9 below:

Table 6: The Value of the B1 Beam Test Parameter at The Time of The Yield and Maximum Load

Parameters measured	Beam Condition	
	Yield	Maximum
Load (P, ton)	14.95	24.30
Moment Maximum (t.m)	198,29	388.80
Deflection (δ, mm)	15.02	68.02
Strain of TensilReinforcement ϕ 9 (ε, μ)	-	-
Strain of shear reinforcement (Sb7) ϕ 10 (ε, μ)	15	25
Strain of spiral reinforcement (Sp5) ϕ 6 (ε, μ)	21	283

Table 7: The Value of the B2 Beam Test Parameter at The Time of The Yield and Maximum Load

Parameters measured	Beam Condition	
	Yield	Maximum
Load (P, ton)	17.62	36.14
Moment Maximum (t.m)	234,32	578.24
Deflection (δ, mm)	15.38	72.08
Strain of TensilReinforcement (B1) ϕ 9 (ε, μ)	1952	4502
Strain of shear reinforcement (Sb9) ϕ 10 (ε, μ)	15	35
Strain of spiral reinforcement (Sp5) ϕ 6 (ε, μ)	79	420

Table 8: The Value of the B3 Beam Test Parameter at The Time of The Yield and Maximum Load

Parameters measured	Beam Condition	
	Yield	Maximum
Load (P, ton)	19.42	32.34
Moment Maximum (t.m)	226,13	517.44
Deflection (δ, mm)	17.09	69.99
Strain of TensilReinforcement (B3) ϕ 19 (ε, μ)	1998	4607
Strain of Shear Reinforcement (Sb7) ϕ 10 (ε, μ)	26	45
Strain of Spiral Reinforcement (Sp5) ϕ 10 (ε, μ)	22	130

Table 9: The Value of the B4 Beam Test Parameter at The Time of The Yield and Maximum Load

Parameters measured	Beam Condition	
	Yield	Maximum
Load (P, ton)	9.93	21.97
Moment Maximum (t.m)	46,13	87.88
Deflection (δ, mm)	9.29	72.19
Strain of TensilReinforcement (B4) ϕ 19 (ε, μ)	2057	9200
Strain of Shear Reinforcement (Sb10) ϕ 10 (ε, μ)	9,0	10
Strain of Spiral Reinforcement (Sp6) ϕ 6 (ε, μ)	187	372

4. Results and Findings

Based on the results of laboratory tests of several precast beam type samples using NK-SpirCon connections, it is known that the maximum deflections and moments that can be assumed by each type of beam are greater than the results of planning calculations using conventional beam (cast in situ). Comparison of the magnitude of these maximum deflections and moments capacity can be seen in Table 10 and Table 11 below:

Table 10: Comparison of NK-SpirCon Beam Deflection and Initial Planning

Beam Type	Dimension (cm)	Deflection (mm)			δ, difference (%)
		δ, laboratory	δ, design	δ, difference	
B1	40 x 60	68.02	47.13	20.89	30.71
B2	40 x 60	72.08	52.44	19.64	27.25
B3	40 x 70	69.99	49.87	20.12	28.75
B4	30 x 50	72.19	51.22	20.97	29.05
B5	40 x 60	65.12	43.45	21.67	33.28
				Average	29.81

Table 11: Comparison of Moment Capacity NK-SpirCon Beams and Initial Planning

Beam Type	Dimension (cm)	Momen Capacity (tm)			M, difference (%)
		M, laboratory	M, design	M, difference	
B1	40 x 60	388.80	287.24	101.56	26.12
B2	40 x 60	578.24	390.56	187.68	32.46
B3	40 x 70	517.44	360.12	157.32	30.40
B4	30 x 50	87.88	46.550	41.33	47.03
B5	40 x 60	239.33	150.28	89.05	37.21
				Average	34.64

5. Conclusion

Based on the above discussion results, this research concludes that the pre-printed beam using NK-SpirCON connection system can withstand deflection and greater capacity moments than the monolith system (cast in situ). The amount of deflection and the capacity moments between precast beams using the NK-SpirCON connection with the conventional beam (cast in situ) system for some beam samples are averaged about 29.81% for deflection and about 34,64 % for the moment of capacity, where precast beam using NK-SpirCON system connection has greater deflection and greater capacity moments.Based on the above discussion results, this research concludes that the pre-printed beam using NK-SpirCON connection system can withstand deflection and greater capacity moments than the monolith system (cast in situ). The amount of deflection and the capacity moments between precast beams using the NK-SpirCON connection with the conventional beam (cast in situ) system for some beam samples are averaged about 29.81% for deflection and about 34,64 % for the moment of capacity, where precast beam using NK-SpirCON system connection has greater deflection and greater capacity moments.

References

[1] Adiasa, A.M. Prakosa, D.K. Hatmoko, J.U.D Santoso, T.D. 2015. Evaluasi Penggunaan Beton Precast di Proyek Konstruksi. Jurnal Karya Teknik Sipil, Vol. 4. No.1, pp 126-134

[2] Tjitrosoma, T.H.R, Subakti, A. 2012. Perancangan Modifikasi Struktur Gedung RSUD. Dr. Kanujoso Djatiwibowo Menggunakan Beton Pracetak dan Metode Pelaksanaan. Jurnal Teknik POMITS, Vol.1, No. 1 (2012), pp. 1-5, Jurusan Teknik Sipil, Fakultas Teknik Sipil dan Perencanaan, ITS Surabaya.

[3] Abduh, M. 2007. Inovasi Teknologi dan Sistem Beton Pracetak di Indonesia ;Sebuah Analisa Rantai Nilai. Seminar dan Pameran HAKI 2007. Konstruksi Tahan Gempa di Indonesia.

[4] Adi, R. Y. Nurhuda, I. Sukamta .Fitriani, I. 2014. Perilaku dan Kekuatan Sambungan Kolom Pada Sistem Beton Pracetak. Media Komunikasi Teknik Sipil. Vol. 20. No. 1, pp. 1-8

[5] Nurjannah, S.A. 2011. Perkembangan Sistem Struktur Beton Pracetak Sebagai Alternatif Pada Teknologi Konstruksi Indonesia Yang Mendukung Efisiensi Energi Serta Ramah Lingkungan. Prosiding Seminar Nasional AVoER ke-3. Palembang, 26-27 Oktober 2011

[6] Wiratman& Associates, 2008, Laporan Perencanaan Struktur Bangunan Gedung Perkantoran dan PerkuliahanTahap II, Fakultas Kedokteran Universitas Wijaya Kusuma Surabaya.

[7] Antonius. 2014. Metode Pelaksanaan Beton Pracetak Pada Struktur Tunnel Feeder. Seminar Nasional Teknik Sipil IV-2014.Jurusan Teknik Sipil, Fakultas Teknik, Universitas Islam Sultan Agung.

[8] Wiranata, A. Ristinah, S. Hidayat, M.T. 2011. Studi Analisis Sambungan Balok-Kolom Dengan Sistem Pracetak Pada Gedung Dekanat Fakultas Teknik Universitas Brawijaya Malang. Thesis (S-2). Program Teknik Sipil Universitas Brawijaya Malang

[9] Syarif, M. Parung, H. Djamaluddin, R. Bakri, A. 2016. Perilaku Sambungan Balok-Kolom Pracetak Type PelatAkibat Beban Bolak-

Balik. Prosiding Seminar Nasional Teknik Sipil 2016. Fakultas Teknik Universitas Muhammadiyah Surakarta.
[10] Anonimus, 2002, Puslitbang Permukiman Pengembang, 2002, "Optimalisasi Sistem Sambungan Komponen Pracetak Struktur Rangka Terbuka untuk Bangunan Bertingkat", Bandung
[11] Bagus, A. Irmawan, M. Faimun. 2013. Analisis DesainSambungan Balok Kolom Sistem Pracetak Untuk RukoTiga Lantai. Jurnal Teknik POMITS Vol. 1, No. 1. Pp. 1-6
[12] Rizal, F &Tavio. 2014. Desain Permodelan SambunganBeton Precast Pada Perumahan Tahan Gempa Di Indonesia Berbasis Knock-down System. JurnalTeknik POMITS, Vol. 3, No. 1. Pp 2301-9271.
[13] Nurjaman , Hari, 2002, "Penentuan Model dan Parameter untuk Analisis dan Perencanaan Tahan Gempa Struktur Pracetak Rangka Beton", Desertasi Doktor Institut Teknologi Bandung, ITB Bandung
[14] Kurniawan, F., Mudjanarko, S. W., & Ogunlana, S. (2015). Best practice for financial models of PPP projects. In Procedia Engineering. Elsivier. https://doi.org/10.1016/j.proeng.2015.11.019