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Experimental Study of Precast Beam Joints Using Steel Plates in Beam-Column Joints Due to Cyclic Loading

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Keywords

beam-column joint; precast concrete; cyclic loading; steel plate.

Abstract

In seismic design, reinforced concrete structures must perform well under heavy load conditions. To withstand large lateral loads without severe damage, structures require strength and energy dissipation capacity. It is generally not economical to design reinforced concrete structures to the greatest possible ground motions without damage. Therefore, the need for ductility must be weighed against economic constraints. This research is focused on dry joints where the joints are on beams in the plastic hinge area and modeled using steel plates, this modeling will be compared with the hysteresis and ductility curves in three types of specimens namely BU-1 without using plate joints, BU-2 using plate joints measuring 300mm x 400mm, BU-3 plate joints measuring 200mm x 400mm, with the same thickness and quality, namely 8mm thick and Fu 370 MPa. To strengthen the use of the plate connection, A325 bolts with a diameter of 12 mm and Fu 620 MPa are used. The results show that the pinching effect is found in the hysteresis curve of the BU-2 and BU-3 specimens. Meanwhile, for the ductility test at peak load, namely the seventh ductility for BU-1, the value is still better than BU-2 and BU-3. These results can still be improved with several variations of plate thickness, for this reason, further studies are recommended to develop and increase recommendations for several variations of plate joints, because precast joints, especially dry joints, are still the focus of research, especially in Indonesia.

1. INTRODUCTION

The use of precast systems for buildings has experienced very rapid development in the world, including in Indonesia in the last decade. Its use is growing rapidly based on several advantages, such as saving processing time, resulting in relatively better efficiency and quality (Elliot, 2016; Wahjudi et al., 2019).

In seismic design, reinforced concrete structures must perform well under heavy load conditions. To withstand large lateral loads without severe damage, structures require strength and energy dissipation capacity. In general, it is not economical to design reinforced concrete structures to the greatest possible ground motion without damage. Therefore, the need for strength and ductility must be weighed against economic constraints. Ductility is an important property of structures that respond inelastically during earthquakes. Ductility is defined as the ability of sections, members, and structures to deform inelastically without excessive degradation of stiffness or strength (KR & GS, 2012).

Joints play an important role in providing structural integrity. Lack of understanding, poor connection design, and neglecting the actual response of joints in structural analysis can lead to inadequate structural design and consequently disaster (Hashim & Agarwal, 2018). This requirement becomes critical in precast construction systems where strength is largely governed by joint capacity. Ellingwood (2007) explained that realistic continuity of load paths between elements through joints can improve structural integrity and stiffness and limit collapse rates.

Beam-column joints are an important element of reinforced concrete structures, especially in earthquake areas. During an earthquake, the beam-column joints are subjected to axial, vertical and horizontal shear stresses through seismic forces, their behavior has a significant influence on the earthquake response of the building causing cyclic motions (Ravikuram & Kothandaraman, 2022; Ahmad et al., 2022; Moehle & Mahin, 1991). Kassem et al. (2022) explained that the lateral forces induced by shear stresses cause diagonal cracks

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in the joints, resulting in shear failure of the joints. Initially, t`he beam reinforcement yielded due to the earthquake force, then the reinforcement in the joint area yielded and bond slip caused a decrease in the strength of the connection.

The ductile properties of a structure can be achieved by considering the stability of the structure during an earthquake. The plastic hinges within a building structure can only occur at the ends of the beams and the column base, and shear walls (Park, 1995; Paulay et al., 1978). Strong connection system can behave elastically in the event of fatigue or collapse outside the joint. Meanwhile, the ductile connection system can experience inelastic deformation. This ductile joint can reduce residual displacement due to drift damage and prevent inelastic deformation (Priestley, 1997; Hawkins & Ghosh, 2000).

Within the elastic boundary where the formation of a plastic hinge will occur in the beam, it is due to its natural ductile behavior. Also, it does not cause global or partial mechanism formation. However, when the plastic hinge is developed in front of the column or the end of the beam, directly related to the joint, it will experience an inelastic joint (Council, 2009). Therefore, the plastic hinge formation should ideally be away from the joint's surface. This is impossible unless special measures are taken to ensure this, making precast systems challenging to develop and inefficient. This efficiency affects the behavior of the joint system, which will transfer the force carried by one part of a structure to another so that the entire construction will bear external force treatment by the behavior of each element. Thus, precast concrete combined with various types of joints must be ductile to lateral and vertical loads (Shariatmadar & ZAMANI, 2014) and to overcome building failure requires the addition of diagonal and straight reinforcement as well as the use of reinforcing thread (Lu et al., 2012; Fernandes et al., 2013) where damage to the common area can be prevented. The earthquake-resistant building regulations of various countries to date still maintain the ductility requirement. Furthermore, beam-column joint reinforcement detail depends on the aspect of detail, strength, and serviceability.

Many experimental and numerical studies were carried out to understand the behavior of beam-column joints due to seismic forces and affect the strength parameters of the joints. Hanson and Connor (Hanson et al., 1967) were pioneers in understanding the behavior of beam-column joints. The experimental results are defined as the horizontal joint shear. Park and Paulay (Park and Paulay, 1973; Park & Paulay, 1991) describe a joint shear retaining system with two mechanisms namely the strut mechanism is the shear force supported by the diagonal section of the concrete to resist the joint shear and the truss mechanism is the shear force supported by vertical and horizontal reinforcement through the bond between the concrete and the reinforcement. after the crack occurred. Other studies have also proposed this mechanism system in predicting joint shear strength and beam column joint behavior (Kim & LaFave, 2008; Yu & Tan, 2014).

In recent decades, several studies have focused on understanding how beam-column joints respond to seismic action (Aycardi et al., 1994; Bracci et al., 1995; Melo et al., 2022). In addition, some earthquake codes have undergone repeated improvements to their implementation. ACI recommended beam-column connection design (ACI-ASCE, 1976) takes inelastic deformation into account. Connection design procedures, detailed proposals for resistance to gravity loads, seismic loads, and the interaction of forces in all directions at the joints of the frame structure. Cyclical behavior and deformation capacity, attachment, and length of reinforcement in beam-column joints.

The precast system at the member connections must be designed in such a way as to have the desired behavior. Structural elements are affected by joints where the connection will transmit the force carried by one part of the structure to another so that the entire construction will be subjected to external force treatment in accordance with the behavior of each element. Then the construction of the connection will occur a complex behavior (Hasan, 2011). So precast concrete which is combined with various types of joints must be ductile against lateral and vertical loads.

Steel plate connection system in several studies, among others. Dessouki et al. (2013) configuration of joint behavior with the ANSYS program in his research identified the effect of plate thickness, bolt diameter, beam height then produced yield line analysis to propose an equation for plate bending capacity. Van-Long at al. (2013) in his test using a circular profile using a steel plate connection to predict the strength of the connection by bending test compared to the results of finite element analysis. Mohamadi-Shoore et al. (2013) the use of hollow profiles with connection modeling using steel plates in his research on the investigation of geometrical characteristics by finite element analysis then produces moment and rotation relationships.

The above research concentrated on steel construction. While research on precast concrete using plate joints including Zhisheng (2013) implementation of precast concrete using plate joints with flexural strength testing and exterior joints the results of his research on cracking, stiffness analysis, energy dissipation. Cheng et al. (2015) applied plate joints to beam-column reinforcement, the results of his research on cracking, hysteresis behavior and ductility analysis.

From the above review, it was concluded that congested areas of reinforcement always create difficulties during erection. In addition, beam-column joints are prone to earthquakes due to bending moments and reverse shear forces. The shear forces induced in the joint area are usually five times the column shear forces (Lin & Restrepo 2002). On the other hand, in frame structures that only adopt beam-column joints, precast components can be very heavy and difficult to transport due to their larger dimensions. With a complete frame configuration of beam-column joints, dense reinforcement details are fabricated. Reinforcement continuity will further enhance integrity and prevent failure (premature failure). Most importantly, the condition where the location of the plastic hinge coincides with the critical joint area can be avoided by requiring joint strength.

This research is focused on dry joints where the joints are on beams in the plastic hinge area and are modeled using steel plates. The principle of stress distribution in beams using steel plate joints has similarities with conventional beams, the force received on the beam, namely the upper reinforcement when compressive loading will be transmitted to the connection through the steel plate. The steel plate will receive force while the shear force is transmitted to the bolt. In the connection the role of the main reinforcement such as a conventional specimen will be replaced by using a steel plate connection, where the plate dimensions are higher than the cross section being joined, the bolt installation is higher, so that the moment-resisting force arm is greater than the moment arm load on the cross-sectional flange plate (Yu & Tan, 2014; Melo et al., 2022). As illustrated in Figure 1. Based on this hypothesis, the behavior of precast concrete beams is expected to be like conventional beams.

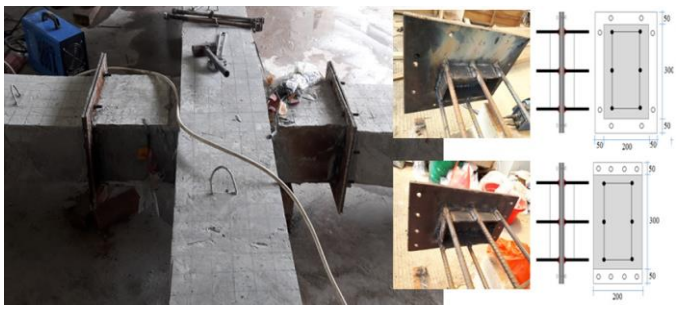


Figure 1. Steel plate joint model

The advantages of using this plate connection, are: (1) Can be machined quickly and safely, significantly increasing the efficiency of the structure; (2) Can optimize the performance of the structure and can be used in the repair of the structure, because the construction and direction of force is clear and simple.

2. EXPERIMENTAL PROGRAMME

2.1 Specimen Design

In this study, the prototype structure was taken from a plane frame structure, the height per floor was 4.0 m, and the distance between columns was 4.0 m. for more details, illustrated in Figure 2.

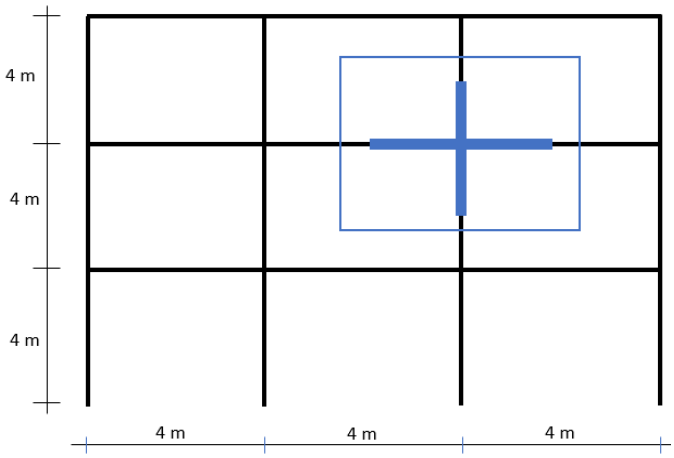


Figure 2. Representation of specimen

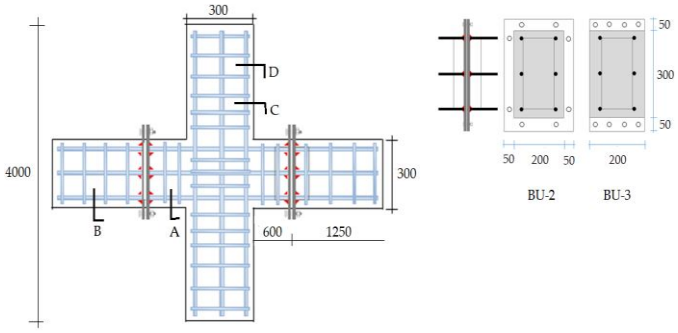


Figure 3. Beam-column dimension and steel plate connection models (BN-2 and BN-3)

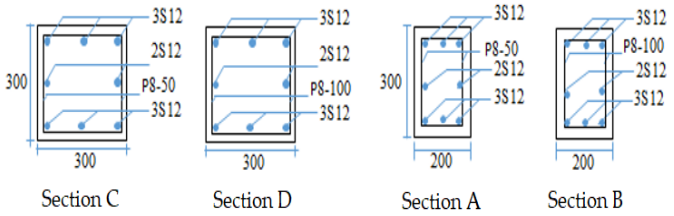


Figure 4. Beam and column reinforcement

The design parameters in figure 3, 4 use three types of test objects, namely BU-1 without using plate joints, BU-2 using plate joints measuring 300 mm x 400 mm, BU-3 plate joints measuring 200 mm x 400 mm, with the same thickness and quality, namely thickness 8mm and Fu = 370 MPa. To strengthen the use of the plate connection, A325 bolts with a diameter of 12 mm and Fu = 620 MPa are used.

2.2 The Specimen Manufacturing Process

In general, the manufacturing process is fully carried out at the Structural Laboratory of Hasanuddin University, Faculty of Engineering, Department of Civil Engineering, Gowa, South Sulawesi - Indonesia. The manufacturing process, as illustrated in Figure 5.

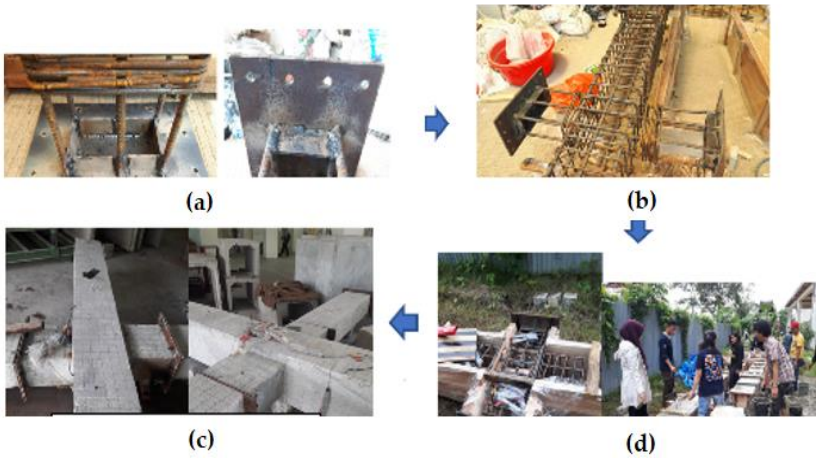


Figure 5. Specimen manufacturing process

Figure 5. explains, a) the steel plate assembly stage, b) the plate is welded with reinforcement before casting, c) the casting stage, and d) the connection to connect precast using bolts.

2.3 Mechanical Properties of Concrete

The mechanical properties of concrete on compressive strength and modulus of elasticity on cylindrical specimens at the age of 28 days can be seen in Tables 1 and 2 as follows.

Table 1. Concrete compressive strength

Description	Plan (MPa)	Result (MPa)
Normal concrete	25	27.40
Grouting concrete		47.19

Table 2. Modulus of elasticity of concrete

Description	Load (N)	Stress (MPa)	Strain
Normal concrete	211760	26.98	4,57 x 10^-4
Grouting concrete	289120	36.83	5,82 x 10^-4

Table 1. Shows that normal concrete quality meets the planned criteria, while grouting concrete is used in joint areas. For Table 2, comparison of the value of stress to strain at the level of 40% concrete compressive strength, the modulus of elasticity for normal concrete is 22780.54 MPa and for grouted concrete is 24638.52 MPa.

3. METHOD AND TEST SETUP

3.1 Method

The test method is based on ECCS (1986), this method describes the procedure system applied, for example, if it is called a complete test procedure (requires three specimens, two for a monotonic test and one for a cyclic test), if it is called a short test procedure (requires only one specimen for cyclic test). For economic reasons, the second procedure is preferred.

Assuming that there is a lack of initial references regarding elastic load (Fy+; Fy-) and elastic displacement (δy+ ; δy-) it is necessary that the displacement step is small enough to ensure that at least four levels of displacement are applied before the elastic displacement is reached. In this procedure the elastic load is defined as the intersection of the modulus tangent (Et+; Et-) on the initial force transfer curve evaluated from the first cycle and the tangent to the envelope curve of the cycle with a slope of 10% from the initial modulus tangent as illustrated in Figure 6.

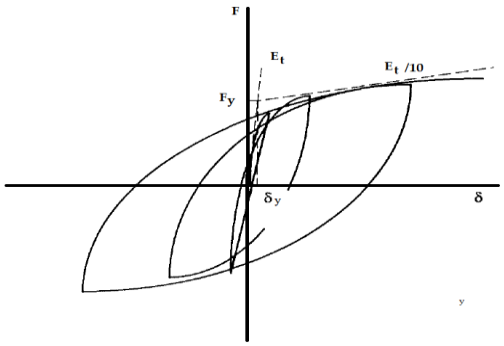


Figure 6. Yield limit reference (ACI Committee, 2005)

The test object is said to perform well if it is fulfilled in both directions of response (ACI Committee, 2005):

- The specimen must achieve a minimum lateral resistance of (En) before its 2% drift ratio exceeds a value consistent with the allowable deviation ratio of applicable seismic regulations.
- The maximum lateral resistance (Emax) recorded in the test should not exceed the value (λEn), where λ is the required overstrength of the test column.
- For cyclic loads at the maximum deviation level that must be achieved as a reference for acceptance of test results, where the value cannot be less than 0.035, the third full cycle characteristic at the deviation level must comply, as follows:
  1. The peak force in the direction of the applied load must not be less than 0.75 (Maximum) in the direction of the same load (figure 7a);
  2. Energy dissipation shall not be less than 1/8 (figure 7b);
  3. The secant stiffness of the line connecting the drift ratio –0.0035 to +0.0035 drift ratio must be not less than 0.05 times the initial stiffness (figure 7c).

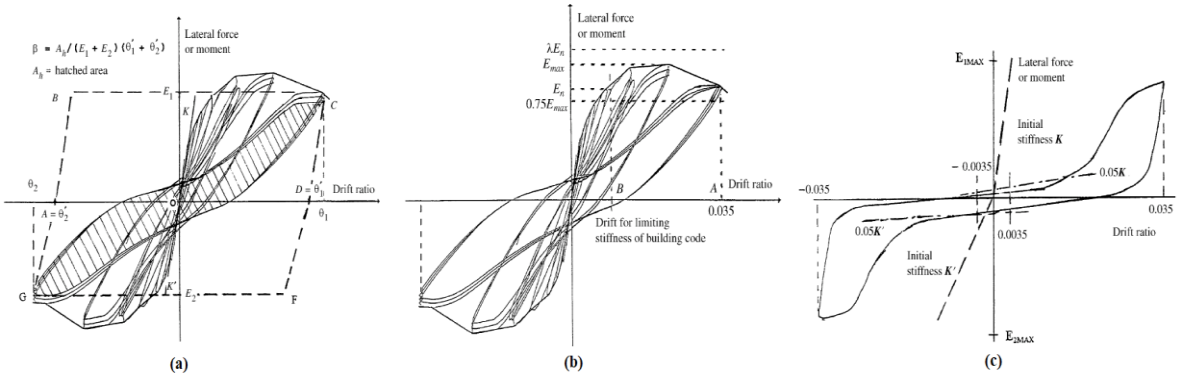


Figure 7. Performance response

3.2 Test Setup

In the ECCS recommendation (1986), which was explained above, if the procedure is simplified, the displacement value (δy) is predicted by the analytical model. Cyclic testing is designed to have a loading pattern as displacement control, as follows:

- One cycle at intervals $\delta_y^+/4$ and δy-/4
- One cycle at intervals $2\delta_y^+/4$ and 2δy-/4
- One cycle at intervals $3\delta_y^+/4$ and 3δy-/4
- One cycle at intervals δ_y+and δ_y^-
- Three cycles at intervals $2\delta_y$+and $2\delta_y^-$
- Three cycles at interval (2+2n) δ_y^+ and (2+2n) δ_y^- (n=1,2..etc)

Then, cycles and intervals are added based on the needs.

The above description explains that the increase in the next cycle is δy/4, 2 δy/4, 3 δy/4, (2+2n)δy, where n=1,2,3, etc. While 2δy(2+2n)δy is planned with short procedure, as illustrated in Figure 8. The ECCS concept is prepared for full cycle loading with tensile and compressive forces as well as positive and negative displacements. The negative displacement of the component under test is outside the accuracy of the transducer. The minimum displacement value is taken as zero, δy = 0. The compressive force in the next cycle returns the specimen to zero displacement after reaching its elastic limit.

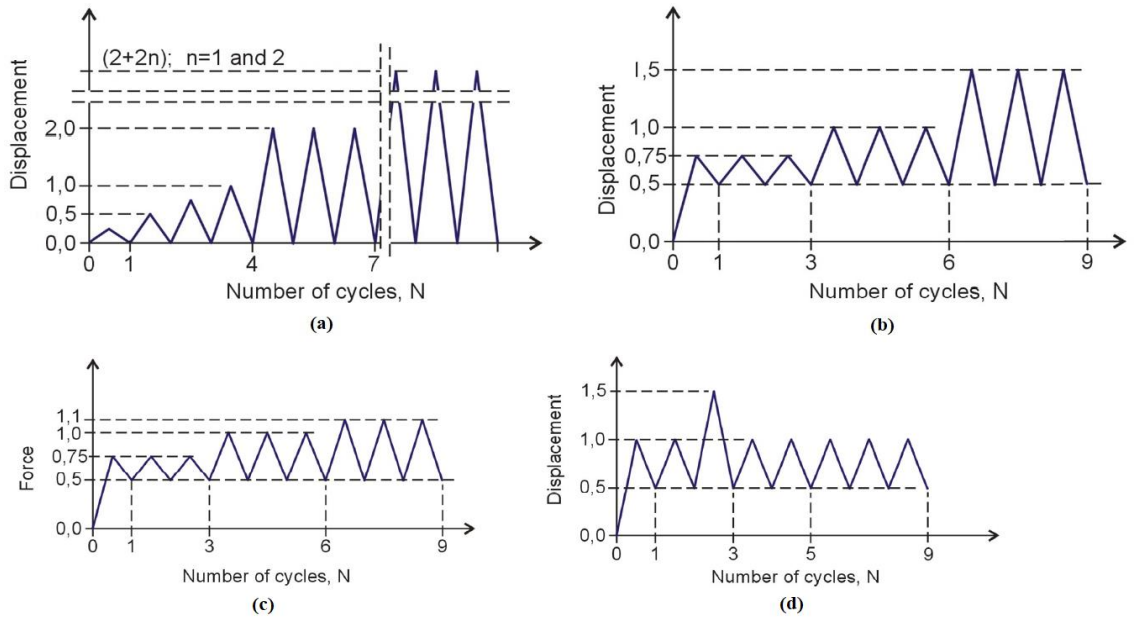


Figure 8. Illstration of a loading pattern (ECCS, 1986; Budziński & Ślęczka, 2020), (a) Loading pattern as recommended by ECCS, (b) Repeated with displacement control, (c) Repeated with load control, (d) Repeated with excess load in the third cycle.

Figure 8(a) it is explained that the record contains several cycles in the elastic range, followed by a group of three cycles of increasing amplitude, with unloading to a zero value with a displacement control pattern. As a reference level, a displacement value corresponding to a characteristic one turn of the screw is accepted.

Displacement references obtained during monotonic loads in each group of specimens tested. Furthermore, it is assumed that the load recording is repetitive, with groups of three cycles whose amplitude increases, with un-loading up to a certain level (not equal to zero). In the case of the record in figure 8(b) loading is controlled by displacement and the degree of unloading is equal to 0.5 at displacement. In the case of the records of figure 8(c) loading is controlled by force and the degree of unloading is equal to 0.5 at displacement, characteristic of the group of specimens under consideration. This load scenario is supposed to simulate a typical load record, where the maximum value of the load is derived from the sum of the permanent and variable actions. The final loading record in figure 8(d) simulates the joint overload in the third cycle outside the elastic range and after that repeated loads with smaller amplitudes.

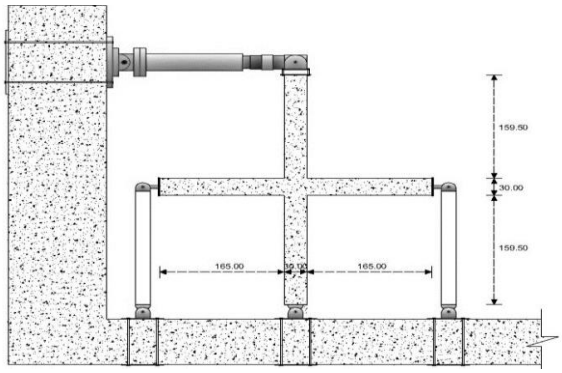


Figure 9. Schematic view (dimensions in mm)



Figure 10. Specimen photo display

Figure 10 can be explained as follows:
- Actuators; used to measure level loads or cyclic horizontal shear forces
- Strain gauges; strain gauges, either installed in reinforcement (Type FLA 6-11) or in concrete (Type PL 60-11)
- LVDT; displacement transducer
- Load cells; force meter circuit
- Longer data; Observe and record the load that occurs

In ECCS [34] divided ductility, calculated by the following equation.

Partial Ductility:

$$\mu_{oi}^{+} = \frac{e_i^{+}}{e_v^{+}} \qquad (1)$$

$$\mu_{oi}^{-} = \frac{e_i^{-}}{e_v^{-}} \qquad (2)$$

Full Ductility:

$$\mu_i^{+} = \frac{\Delta e_i^{+}}{e_v^{+}} \qquad (3)$$

$$\mu_i^- = \frac{\Delta e_i^-}{e_y^-} \qquad (4)$$

where:

e_i^-, e_i^+ is limit deviation.
Δe_i^-, Δe_i^+ is total of limit deviation.
e_y^-, e_y^+ is deviation in the yield condition.

The above equation can be represented in the form of a curve, as illustrated in Figure 11 below.

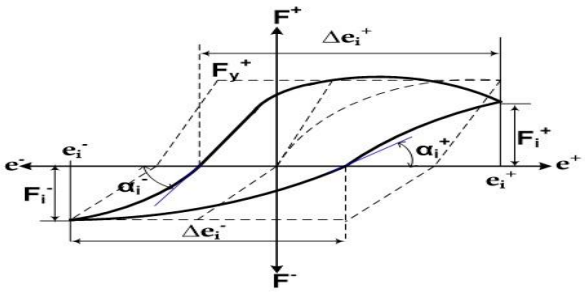
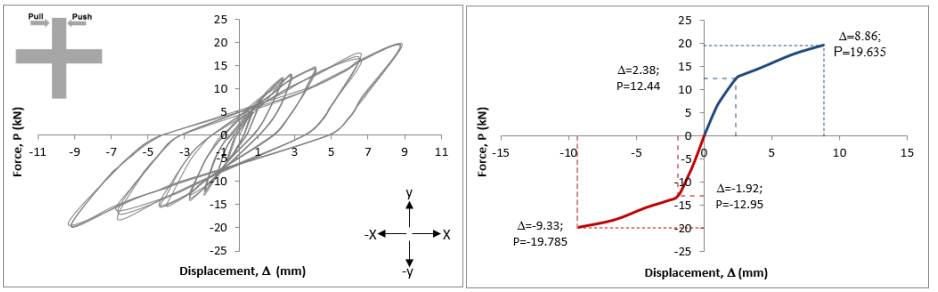


Figure 11. Value of the characteristic moment-displacement curve

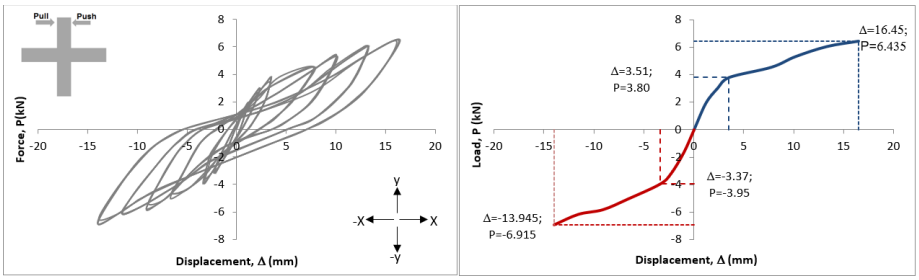
4. RESULTS AND DISCUSSION

4.1 Hysteresis Curve

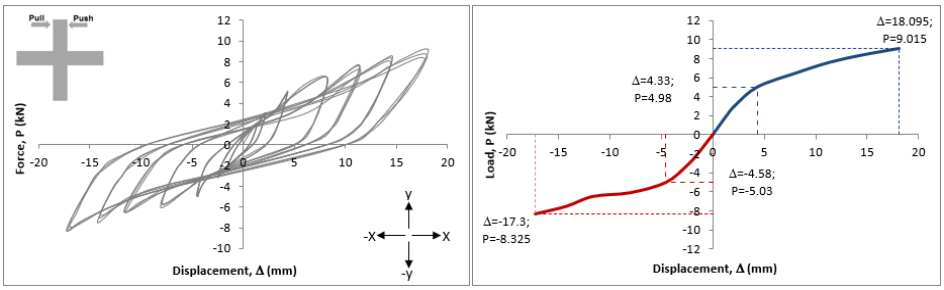
The hysteresis curve, which is illustrated in Figure 12. The x axis represents displacement, and the y axis represents loading. The peak line of the first cycle on the hysteresis curve per cycle of load is the skeleton curve.



(a) (b)



(c) (d)



(e) (f)

Figure 12. Hysteresis curve; (a) Hysteresis curve for BU-1; (b) Skeleton curve for BU-1; (c) Hysteresis curve for BU-2; (d) Skeleton curve for BU-2; (e) Hysteresis curve for BU-3; (f) Skeleton curve for BU-3.

Based on the hysteresis curve in Figure 12. Shows a significant difference compared to the test object (BU-1). In addition, the pinching effect on the two specimens (BU-2 and BU-3) where the greater the deformation, the smaller the ability to withstand loads, as well as the ability of the specimens (BU-2 and BU-3) to carry loads on the plastic condition is smaller when compared to the test object (BU-1). This is not in accordance with the plans and hypotheses that have been made. The precast specimen does not behave like a monolithic structure which can be deformed collectively. The dominant behavior that occurs is the movement of the beam against the joint on the surface of the steel plate, when a horizontal force is applied to the column, the behavior of the movement of the steel plate joint on the beam forms a rotational deformation.

4.2 Ductility for Beam-Column Joints Test

Ductility is the ability of a structure to undergo repeated alternating inelastic deformations beyond its first yield point, while retaining a large amount of its load-bearing capacity. Data analysis refers to the ECCS standard.

From equations 1 and 2, the partial ductility results are presented in tabular form, as follows.

Table 3. Partial Ductility for BU-1

Ductility number	Push			Pull			
	e_i^+ (mm)	e_y^+ (mm)	m_{oi}^+	e_i^- (mm)	e_y^- (mm)	m_{oi}^-	$m_{average}$
1	0.72		0.30	-0.70		0.36	0.33
2	1.17		0.49	-1.20		0.62	0.56
3	2.38		1.00	-1.92		1.00	1.00
4	2.84	2.36	1.19	-2.73	-1.92	1.42	1.31
5	4.16		1.75	-4.37		2.28	2.01
6	6.64		2.80	-6.65		3.46	3.13
7	8.86		3.73	-9.33		4.86	4.29

Table 4. Partial Ductility for BU-2

Ductility number	Push			Pull			
	e_i^+ (mm)	e_y^+ (mm)	m_{oi}^+	e_i^- (mm)	e_y^- (mm)	m_{oi}^-	$m_{average}$
1	1.18		0.33	-1.02		0.30	0.32
2	2.34		0.34	-2.29		0.68	0.67
3	3.51		1.00	-3.37		1.00	1.00
4	7.76	3.51	2.13	-6.64	-3.37	1.97	2.09
5	10.14		2.89	-9.09		2.97	2.79
6	13.30		3.79	-		3.42	3.61
7	16.45		4.69	-		4.14	4.42

Table 5. Partial Ductility for BU-3

Ductility number	Push			Pull			
	e_i^+ (mm)	e_y^+ (mm)	m_{oi}^+	e_i^- (mm)	e_y^- (mm)	m_{oi}^-	$m_{average}$
1	1.34		0.31	-1.46		0.32	0.31
2	2.15		0.49	-2.49		0.54	0.52
3	4.33		1.00	-4.58		1.00	1.00
4	8.20	4.33	1.89	-8.04	-4.58	1.76	1.82
5	11.40		2.63	-		2.55	2.59
6	14.59		3.37	-		3.12	3.24
7	18.10		4.18	-		3.78	3.98

The partial ductility above, specimen of BU-1, BU-2 and BU-3, gives an upward graph, starting from the 1st to the 7th ductility. The largest value was given to BU-2 which gave a yield of 4.42, compared to BU-1 which was 4.29, while BU-3 experienced a decrease in value of 3.98.

The full ductility test is given in the following table.

Table 6. Full Ductility for BU-1

Ductility number	Push			Pull			
	D_{ei}^+	e_y^+	m_i^+	Dei^-	e_y^-	m_i^-	$m_{average}$
	mm	mm		mm	mm		
1	0.73		0.30	-0.86		0.44	0.38
2	1.45		0.61	-1.87		0.82	0.71
3	2.83		1.19	-2.63		1.37	1.28
4	3.67	2.38	1.54	-3.37	-1.92	1.95	1.75
5	5.65		2.38	-5.81		3.03	2.70
6	9.74		4.10	-		5.25	4.68
7	13.00		5.47	-		7.30	6.39

Table 7. Full Ductility for BU-2

Ductility number	Push			Pull			
	D_{ei}^+	e_y^+	m_i^+	Dei^-	e_y^-	m_i^-	$m_{average}$
	mm	mm		mm	mm		
1	1.32		0.38	-1.37		0.40	0.39
2	2.64		0.75	-2.83		0.89	0.80
3	4.36		1.24	-4.52		1.34	1.29
4	9.83	3.51	2.80	-8.87	-3.37	2.63	2.72
5	13.44		3.83	-		3.77	3.80
6	17.40		4.96	-		4.96	4.96
7	21.77		6.21	-		6.25	6.23

Table 8. Full Ductility for BU-3

Ductility number	Push			Pull			
	D_{ei}^+	e_y^+	m_i^+	Dei^-	e_y^-	m_i^-	$m_{average}$
	mm	mm		mm	mm		
1	1.79		0.41	-2.08		0.45	0.43
2	3.01		0.69	-3.47		0.76	0.73
3	5.85		1.35	-6.79		1.48	1.42
4	12.40	4.33	2.86	-	-4.58	2.90	2.88
5	17.12		3.95	-		3.91	3.93
6	21.82		5.04	-		4.95	4.99
7	27.40		6.33	-		6.00	6.16

Full ductility as shown in table 6, 7, and 8 increases in each loading cycle from cycle 4 to cycle 6, while in cycle 7 there is a decrease.

From the two data and curves above, it can be concluded that even though there is a pinching effect and there is movement at the plate joints which results in rotational deformation, they are still able to maintain their bearing capacity. Based on FEMA 356 table 6.6 (FEMA, 2000), the maximum value for displacement ductility is $\mu > 4$, so fall into the category of High Ductility demand.

5. CONCLUSION

Based on the results of the analysis and discussion, it is concluded as follows:

1. The hysteresis curve of the load-displacement relationship on the precast test object using the resulting plate shows a very significant difference to the monolith test object (BU-1), where the maximum load BU-1 produced is an average of 19.71 kN, the test object precast (BU-2) of 6.68 kN, while precast specimen (BU-2) of 8.67 kN.
2. The occurrence of a pinching effect causes degradation of strength and stiffness due to the large displacement mechanism, namely BU-1 the average displacement is 9.10 mm, BU-2 is 15.20 mm, while BU-3 is 17.70 mm, this indicates that the load capacity can be withheld very small as described in point 1.
3. At the level of partial ductility for BU-1 the average yield obtained is $\mu = 4.29$, BU-2 is $\mu = 4.42$, and BU-3 is $\mu = 3.98$.
4. At full ductility level for BN-1 the average results obtained are $\mu = 6.39$, BU-2 is $\mu = 6.23$, and BU-3 is $\mu = 6/16$.

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