Shear Behavior of Joint The Partial Prestressed Concrete Beam-Column Reinforced Concrete of Ductile Frame Structure Building In a Scure Residents and for Settlement Environment

Made D Astawa[1], IGP Raka[1], and Tavio[1]

Abstract—***Concentration of this study is to create a Specimens model of Joint Interior Beam - Column using Partial Prestressed Concrete Beams elements connected with Reinforced Concrete Column. Design capacity of Beam- Column Joint Shear is the horizontal shear reinforcement in the form of stirrups ∅10-50 mm to fill the empty space bj = 288 mm . The capacity Shear that can be deployed by the cross bar = 103.62 kN. Total shear force that is capable of detained by the beam-column joint structures are Vjh = 409 kN. This study is a continuation of research SRPMK models shaped beam - column joint with beam section 250/400 mm , and the column section 400/400 mm , the source of funds from Research Ditlitabmas Decentralization Program of Higher Education , through ITS Featured Research Grant in 2013 . Experimental studies have been conducted with Cyclic loading (pseudo dynamic) lateral , static axial load on the column as a stabilizer . Specimens ability to withstand Ultimate Lateral Cyclic Load : conditions Load push (press) = 470.90 kN and Load Pull = 465.80 kN. Everything is > 409 kN. Ductility structure also qualified in 3.50% Drift Ratio: Conditions Press μ = 1.27 > 1.20, Pull Conditions μ = 1.29 > 1.20. In general, behavioral modeling structure has qualified as a reliable protection occupancy when the building was hit by an earthquake.***

Keywords—The Beam-Column Joint shear Behavior, ductile, Earthquake Resistant.

I. Introduction

Investigation of the collapse of the post-quake buildings in areas hit by the earthquake lately, such as the earthquake in Aceh (2004), Yogyakarta (2006), West Sumatra (2009), mostly due to a design fault structural elements beam-column Joint, because the strong-column design principles are lacking weak beams, namely the lack of transverse reinforcement as a barrier shear reinforcement in columns, precisely at the beam-column joint. So in this study will design a retaining structure element model of the quake at the confluence ductile reinforced concrete column-partially prestressed concrete beams reliable against shear failure and bending, as the protection of a safe and comfortable shelter.

II. The Fundamental Theory

A. *Load-deflection relationship on prestressed concrete.*

Load-deflection relationship for beams with some variation of prestressing force is presented in Figure 1. Both under-reinforced concrete beams or over-reinforced concrete beam[16].

B. *Flexure tension Elastic and Partial Prestressed Beams strength after cracking.*

At the stage of full service load, partial prestressed beams are usually cracked, although stress concrete and steel stress remains in the elastic range. Even more complicated is for cracked prestressed concrete beams. Axial force is not constant after the crack, but depending on the loading and the nature of the cross-section. Effective cross-section of a typical partial prestressed beams at serviceability load is shown in Figure 2. (a). Load step (1) Figure 2. (b) application effective prestressing force Pe only. At this stage,the stress in the tendon is :

$$\boldsymbol{f_{p1} = f_{pe} = \frac{P_e}{A_p}} \quad (1)$$

In the compressive strain assumed perfect bonding between the two materials, similar to those seen at the same level concrete. Thus, reinforcement is subject to compressive stress initials :

Furthermore, fictitious load stage (2), is useful to consider the stage of the fictitious load corresponding to complete decompression of the concrete, the concrete strain zero through the entire depth (Figure 2. (c)). Deformation compatibility of concrete and steel require Stress changes in the tendon and reinforcement rods on each beam to move from step (1) to step (2):

$$f_{p2} = E_p \,.\, \varepsilon_{p2} \quad (2)$$

$$f_{s2} = E_s \,.\, \varepsilon_{s2} \quad (3)$$

At this stage hypothetical load, stress on the rod reinforcement, ignoring the effect of shrinkage and creep, are :

$$f_s = E_s(-\varepsilon_{s2} + \varepsilon_{s2}) = 0 \quad (4)$$

Stress change in the same tendon that is in the concrete at that level, and can be calculated based on the properties of concrete has not cracked section :

$$\boldsymbol{\varepsilon_{p2} = \frac{P_e}{A_c \,.E_c}\left(1 + \frac{e^2}{r^2}\right)} \quad (5)$$

after fp2 can be calculated from Eq. (2.4). Reinforcement rods without stress in step (2), but to produce a zero voltage condition on the concrete, tendon to be pulled by external forces fictitious by :

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$$F = A_p(f_{p1} + f_{p2}) \quad (6)$$

is shown in Figure 2. (c). Effect of compressive force is now postponed to give fictitious force F is equal and opposite, as shown in Figure 2. (d). This style works together with external moment M, due to its own weight and load superposition, can be represented by a resultant force R applied with eccentricity e¯above the neutral line is not cracked concrete, where R = F and :

$$\bar{e} = \frac{M_t - F_e}{R} \quad (7)$$

Beam can now be analyzed as ordinary reinforced concrete members were eccentric compressive force. Resultant strain distribution in concrete (3) is shown in Figure. 2. (e). An increase in strain in the tendons and reinforcing rods, and ε ε p3 s3, respectively, together with the corresponding stress FP3 and FS3, imposed on stress and strain existing in the tendon and reinforcement rods.

The addition of the steel stress, and stress in the concrete, can be searched by using the concept of cross-section transformation. Tendons can be replaced by a broad cross-section of tensile eqvalen npAp concrete and reinforcing rods were replaced by broad nsAs, where $n_p = \frac{E_p}{E_c}$ dan $n_s = \frac{E_s}{E_c}$, as shown in Figure 3(a). Neutral axis for transformation eqvalen homogeneous cross section, with the distance y from the surface, can be found from the moment equilibrium condition due to internal force around the entire work line action R must be zero. Compressive stress in concrete due to internal forces and the compressive stress and the action transforming on s teel section, as shown in Figure 3 (b).Moment equations for the internal force of the resultant external R generate the cubic equation for y that can be solved through a trial (trials). Once the magnitude of y, the transformation of the effective area and moment of inertia ektif Act Ict of cracked cross section is known, on neutral axis within c * 1 of the top surface of the cross section. Attempts to raise the tension,escaped from loading phase (2)to step (3), is :

$$\boldsymbol{f_{c3}} = -\frac{R}{A_{ct}} - \frac{R_e^* . c_1^*}{I_{ct}} \quad (8)$$

$$\boldsymbol{f_{p3}} = \boldsymbol{n_p}\left[-\frac{R}{A_{ct}} + \frac{R_e^*(d_p - c_1^*)}{I_{ct}}\right] \quad (9)$$

$$\boldsymbol{f_{s3}} = \boldsymbol{n_s}\left[-\frac{R}{A_{ct}} + \frac{R_e^*(d_s - c_1^*)}{I_{ct}}\right] \quad (10)$$

where the geometric requirements as defined in Figure 3 end tension of the tendon is now obtained by superposition of the stress equation (2.1), (2.3), and (2.10). tension in the reinforcement bars is given by equation (2.11). Concrete tension on the upper surface of the beam is given by equation (2.9). In particular:

$$f_p = f_{p1} + f_{p2} + f_{p3} \quad (11)$$

$$f_s = f_{s3} \quad (12)$$

$$f_c = f_{c3} \quad (13)$$

C. Beam-Column Joint

Test on joint and beams have shown that the shear strength is not sensitive to the shear reinforcement along the span. Then the ACI Code (1) assumes joint force only as a function of the compressive strength of concrete that requires a minimum amount of transverse reinforcement in the joint. Aj, in the Figure 4 of ACI 318 comments should not be> Ag section the column. Minimum shear strength of the joint should not be> Vn specified below-normal weight for concrete.

1. Restraint on all blocks that assemble into columns in front of the joint :

$$V_n \le 20\sqrt{f'_c} . A_j \quad (14)$$

2. Restraints in the three face or two faces opposite columns :

$$V_n \le 15\sqrt{f'_c} . A_j \quad (15)$$

3. All other cases :

$$V_n \le 12\sqrt{f'_c} . A_j \quad (16)$$

Aframework of beams considered to provide confinement to the joint only if at least three quarters of the joint is covered by the beam. Vn value is allowed to be reduced by 25% if used in lightweight concrete. In addition, the test data shows that the value of equation (2.17) is not conservative when applied to the joint angles. Aj = effective cross-sectional area in the joint, as shown in Figure 4, the condition of flat parallel to the plane of shear reinforcement on the average produce joint. ACI regulation assumes that the horizontal shear in the joint is determined on the basis that the flexural tensile stress in the steel fy = 1.25. Figure 5 shows the forces acting in the the beam-column relations in the the joint.

D. Reinforcement distribution on the Joint

For reinforcement bar sizes 3 to No. 11 ends in a joint exterior with standard hooks 90 ° on the normal concrete, length delivery outside face of the column ldh, as required by ACI 318 regulations, shall not be less than the value of the largest of the equation (2.18) and (2.19) and (2.20) the following :

$$l_{dh} \ge \frac{f_y . d_b}{\left(65\sqrt{f'_c}\right)} \quad (17)$$

$$l_{dh} \ge 8\ d_b \quad (18)$$

where db = bar diameter.

$$l_{dh} \ge 6 \text{ inch} \quad (19)$$

Length distribution given out advance column should not be less than ld = 2.5 ldh when the depth of the concrete cast in one ride down the slope reinforcement exceeds 12" All straight bar ends on the joint reinforcement required for confinement passes through the core of a column or shear wall boundary rods. Every part, no longer restrained planting in the core must be increased by a factor of 1.6.

E. State of the Art Shear ductility of Beam-Column Joint

Uma, S.R & Meher Prasad. A (2006)

They conducted a joint study at seismic behavior of reinforced concrete frame beam-column moment bearers (25). The aspects studied include: force of action at the beam-column joint, frame and pedestal respected contribution mechanism at joint, bonding requirements, factors affecting the bonding strength, at joint shear requirements.

In terms of shear at joint analysis, more detail is described as follows:

1. Shear force at beam-column joint Interior

Note assembling parts of interior beam-column joint extends between the points of-counter bending, as shown in Figure 6 (a). Shear forces acting on the joint can be calculated by using the criteria of balance. High center to the center of the column is lc and range of center-to-center beam is lb. Figure 6 (b) shows the strength of the beam joint work in advance. Bending moment and shear force distribution for each column is shown in Figure 6 (c) and Figure 6 (d). To read Figure 6 (c) it is clear that the nature of the moment above and below the joint changes and shows a steep gradient in the joint, causing a large shear forces in the joint compared with that in the column. Horizontal shear force across the joint can be obtained based on the criteria of balance. See arch bending moments, moments Ms and Mh work on the advance with the opposing forces on the joint between the beams are stringing. Assuming symmetrical reinforced beams, tensile force Tb and compressive force Cb done in reinforcement beams. Slide the vertical beam on the face of the joint is Vb. Assuming the shear force Cb = Tb, slide on the columns = Vcol, from forces above is calculated as the equilibrium criterion.

$$\mathrm{V_{col}} = \frac{2\mathrm{T_b}.\mathrm{Z_b}+\mathrm{V_b}.\mathrm{h_c}}{\mathrm{l_c}} \quad (20)$$

wherein:

lc = height of the floor (the Figure 7 (a).

hc = height of the column.

Zb = the lever arm.

Given the slope of the moment in the joint core, horizontal shear force, vjh can be written as :

$$\mathrm{V_{jh}} = \mathrm{V_{col}}\left(\frac{\mathrm{l_c}}{\mathrm{Z_b}} - 1\right) - \mathrm{V_b}\left(\frac{\mathrm{h_c}}{\mathrm{Z_b}}\right) \quad (21)$$

1) The Joint Shear strength

Joint shear strength is strongly influenced by the parameters that influence the two principles against sliding mechanism. Total force contributed by each mechanism can be considered as the shear strength of the joint in the horizontal direction is calculated by :

$$\mathrm{V_{jh}} = \mathrm{V_{ch}} + \mathrm{V_{sh}} \quad (22)$$

where Vch is the contribution of the concrete strut and Vsh is a contribution of the truss mechanism. Contribution of each mechanism is influenced significantly by the prevailing conditions of the bond as discussed in the previous section. Of reference the results this research, the idea arose to investigate the shear capacity of the joint partially prestressed concrete beams with reinforced concrete columns, shear ductility in particular reliability, to avoid storey frame structure of shear failure due to lateral seismic loads. This study is the continuity of the previous year studies that have examined about bending ductility of the structure model of the same order. State of the Art this research is to explore and find the idea of the results of previous studies that the researchers "Column-Behavior Relations Slide Concrete Beams on the Framework Strukutr Daktil as Environmental Building a Reliable and Safe Housing"

III. Research Methodology

A. Research Design

The manufacture SpecimenConcrete compressive strength f'c plan f'c= 40 Mpa, quality steel fy = 400 MPa, fy prestressing steel quality ≥ 1000 MPa. Preliminary test objects including concrete and tensile steel tendons, have been made in the implementation of research years ago. So in this study is planned immediately make Beam-Column Joint interior structure element model consisting of partially prestressed beam elements and elements of non-prestressed reinforced concrete columns with shear reinforcement in accordance with the design results. Beam section dimensions of 250/400 mm, column section 400/400 mm. Specifications of the test objects are arranged in the following table 1.

The draft results of specimens shaped Beam-Column Joint interiors that are resistant to earthquake lateral shear forces such as sketch the following Figure 9.

B. Shear reinforcement Design in the joint

Actuator = 1000 kN capacity, efektive 80% = 0.8(1000)=800kN. For the design load capacity of the structure, all the specimens were taken into account in the structural condition of the elastic condition, so the structure has not been cracked. For Beam-Column Joint specimens Interior:

1) The actuator moment due to lateral force P: 800 kN.(1,00 m) = 800 KNM, a primary moment. Members stiffness:

$$\mathrm{k_{1,3}} = \frac{3\ \mathrm{EI_1}}{\mathrm{L_1}} = \frac{3.\frac{1}{12}.25.40^3}{160} = 4{,}0$$

$$k_{2,4} = \frac{3.EI_2}{L_2} = \frac{3.\frac{1}{12}40.40^3}{130} = 6{,}4;$$

Moment distribution factors :

$$\mathrm{fd_1} = \mathrm{fd_3} = \frac{\mathrm{k_1}}{\mathrm{k_1+k_2+k_3+k_4}} = \frac{4}{4+6{,}4+4+6{,}4} = 0{,}19$$

$$\mathrm{fd_2} = \mathrm{fd_4} = \frac{\mathrm{k_2}}{\sum \mathrm{k}} = \frac{6{,}4}{4+6{,}4+4+6{,}4} = 0{,}31$$

$M_1 = M_3$ = 0,19 (800) = 152 kNm ; M2 = M4 = 0,31 (800) = 248 kNm

Mn1 reinforced bar = 102,54 kNm

Mn of strand tendon :

X = a/β_1 = 82,4/0,77 = 107 mm ; e = 282-107 = 175 mm

Mn2 = F (e) = 379 (175).10^{-3} = 66,33 kNm

Mn=Mn1+25%(Mn2)=85,96+0,25(66,33)=102,54 kNm < 152 kNm (OK).

Calculate the Strong Column-weak beam requirement :

Me = 248 kNm ; Mg = 102,54 kNm

Requirements of SNI 03-2847-2002 part 23.4.2 (2) :

Me≥6/5Mg, accordingly $248 \geq {}^{6}/_{5}\ (102{,}54) \rightarrow 248\ \mathrm{kNm} > 123{,}05\mathrm{kNm(OK)}$

C. Shear Stirrups Design in the Joint

According to the provisions of SNI 03-2847-2002 part 23.4.4) :

$$\mathrm{A_{sh}} = 0{,}3\left(\frac{\mathrm{s.h_c}.f_c'}{\mathrm{f_{yh}}}\right)\left[\left(\frac{\mathrm{A_g}}{\mathrm{A_{ch}}}\right) - 1\right] \quad (23)$$

$$\text{Atau}\ \ \mathrm{A_{sh}} = 0{,}09\left(\frac{\mathrm{s.h_c}.f_c'}{\mathrm{f_{yh}}}\right) \quad (24)$$

stirrups ∅10, 40 mm concrete cover
Ach = (320)(320) = 102400 mm^2, h_c = 320 – 2(0,5. 10) = 310 mm. Stirrup spacing S is taken 50 mm.

$$A_{sh} = 0{,}3\left(\frac{50.310.40}{400}\right)\left(\frac{160000}{102400} - 1\right) = 261{,}56\ \mathrm{mm}^2$$

or $A_{sh} = 0{,}9\left(\frac{50.310.40}{400}\right) = 139\ \mathrm{mm}^2$,used a great value.

$1\emptyset10 \rightarrow As = 78{,}5\ \mathrm{mm}^2$, the amount of stirrups =261.56/78.5=3.33 it takes 4 stirrups, but because S = 50 mm, and height of the beam space = 400-2(35)-2(8)-2(13) = 288 mm, we used amount of stirrups = 288/50 = 5,76 ; 6 pieces rounded stirrups with A_{sh} = 471 mm^2.

D. Calculating the shear strength of joint

Above beam reinforcement is 5D13 with As = 663.7 mm^2. 3D13 reinforcement in bottom beam with As = 398.2mm^2. Effectively the high block requirements for concrete compressive stress is partially prestressed concrete $a = 0{,}235d \leq 0{,}2h\ s/d\ 0{,}25\,h$, then taken: $a = 0{,}235\ (350{,}5) = 82{,}4\ \mathrm{mm}\ \leq 0{,}25.400 \rightarrow$ $82{,}4\mathrm{mm} < 100\ \mathrm{mm} \ldots (\mathrm{OK})$
As-as Column high = 3,0 m

$$V_{col} = \frac{2T_b.Z_b + V_b.h_c}{l_c} \text{ from the equation 20}$$

$$V_{jh} = V_{col}\left(\frac{l_c}{Z_b} - 1\right) - V_b\left(\frac{h_c}{Z_b}\right) \text{from the equation 21}$$

V_b = 0,4. 0,25. 24(1,6) + 192/1,6 = 123,84 kN

$$V_{col} = \frac{102{,}54 + (398{,}2.400)\left(365 - {}^{82{,}4}/_{2}\right)10^{-6} + 123{,}84(0{,}4)}{3{,}0} = 68\ \mathrm{kN}$$

$$V_{jh} = 68\left(\frac{3{,}0}{0{,}3238} - 1\right) - 123{,}84\left(\frac{0{,}4}{0{,}3238}\right) = 409\ \mathrm{kN}$$

Shear strength at the joint are calculated of the nominal strong concrete and reinforcement in the joint stirrups

$V_{jh} = V_{ch} + V_{sh}$ from the equation 22

$V_n \leq 12\sqrt{f'_c}.A_j$ from the equation 16

The equation 2.17 is applicable for Beam-Column Joint frame field. High joint taken = Column high = 400 mm, the effective width taken of the smallest value: beam width (b), or b + 2X; X = difference thick outer edge of the beam to the column. b+h = 250 + 400 = 650 mm; b+2X = 250 +2(75) = 400 mm, taken effective width = 325 mm. then Aj = 400(325) = 130000 mm^2.

$$V_n \leq 12\sqrt{40}.130000.10^{-3} = 9866{,}3\ \mathrm{kN}$$

Reduction factor of 0.55 according to SNI 03-2847 section 11.3 (3) (a) for the structure to withstand earthquake forces.Vch=ϕ.Vn=0,55(9866,3)=5426,5 kN
Vsh = ϕ.Ash. f_{yh} = 0,55(471)400.10^{-3} = 103,62 kN

$V_{jh} = V_{ch} + V_{sh}$ = 5426,5 + 103,62 = 5530,12 kN ... (OK)
But the decisive factor is the smallest vjh = 409 kN.

E. Test specimens in laboratory.Test specimens was performed by laboratory testing machine, where the specimens were installed tool

Linear Variable Displacement Transducer (LVDT) on the vertical and horizontal displacement to measure deformation (displacement) that occurred. To detect strains occurs in both the beam or the column, then at certain points-installed strain gauge. Loading pattern is Cyclic loading pattern (pseudo dynamic) that resembles a real earthquake lateral loads, driven actuator with a capacity of 2000 kN. For the vertical load on the column is static loading capacity of 1000 kN.

F. Specimen test results expected

a. There was good cooperation between the concrete reinforced strand tendons in response to the earthquake Cyclic lateral shear loads, so vjh ≥ Vuh both conditions are still elastic nor the inelastic conditions.
b. Shear ductility Vn ≥ Vu of the joint, until the boundary load is done through a horizontal in laboratory experiments.
c. When the shear capacity of the specimens fulfilling Vn ≥ Vu, then the ductility $\mu = (\delta_{max} / \delta_{\text{first yield}})$, will also be fulfilled.

IV. Experimental Analysis of Test Results in Laboratory

To get accurate data from Beam-Column Joint research is then mounted several sensors at the points that are important to the tool, including the form; Linear Variable Displacement Transducer (LVDT), Starain-gauge (SG) and Wire-gauge (WG). Each outcome data at every point in the form of graphs will be presented sequentially.

Results analysis Test Specimens at peak Interior Column. For the beam-column joint specimens Interior, the Static Axial load on a given column by vertical actuator load capacity by 10% Column = 10% (400x400) 40 X 10-3 = 640kN. Load data will be shown in the following table 2 for conditions on top of the column.

A. Load Structure resist capability at Joint Shears from reading the data starin-gauge (SG) in a row

SG-13 (reinforcement columns) + SG-16 (Sengkang Scroll) + SG-25 (Strand Tendons) + SG-40 (Concrete Column). Lateral Press Forces = 142.3 +142.3 kN +44.3kN+142=470.9kN.
Lateral Pull Forces = 135 kN kN +138 +58.5 +134.3 kN=465.8kN.
Vjh plan = 409kN, the ability resist lateral test force results of Specimen : Press = 470.9 kN and Pull = 465.8 kN. All of them > 409 kN ... (OK)

B. Load capacity compared Load Melt Early specimens in accordance with SNI 03-1726-2002

$P_y/P_i \geq f_i$, where f_i = 1.2, Ideal load (Pi) conditions on the elastic Story Drift 1,0 % the first cycle :
P_i (press) = 108, 50 kN ; P_i (pull) = 106,20 kN
Maximum load (Py) yield conditions on Story Drift 3,5% 13rd cycle :
P_y(press) = 138,30 kN ; P_y (pull) = 137,60 kN.
$P_y/P_i \geq 1{,}20$ 138,30/108,50 ≥ 1,20 (press) ⟶
μ =1,27 > 1,20 ………...(OK)
$P_y/P_i \geq 1{,}20$ 137,60/106,20 ≥1,20 (pull) ⟶
μ = 1,29 > 1,20 …………(OK)

C. Structural stability

There are 3 criteria that must be met by the specimen in accordance with the "Proposed Revision to 1997 NEHRP Recommended Provisions for Seismic Regulations for Precast Concrete Structure" and the American Concrete Institute (ACI). Analysis of test results according to the 3 criteria are:

Load capacity of Specimen to Maximum working Cyclic Lateral Load..

1) The maximum load at 4.50% Story Drift :
Press = 142,30 kN
Pull = 135,70 kN.
StoryDrift in 3rd Cycle:
Lateral Press Load on the 3rd Cycle = 114.80 kN. Lateral Pull Load on the 3rd Cycle = 99.30 kN. So that the load carried by the Specimens is able to: Load Press = 114.80/142.30 (100%) = 80.60% > 75%…(OK)
Load Pull = 99.30 / 135.70 (100%) = 73.17% <75% (not OK)

2) The maximum load at Story Drift : 3,50 % Cycle 3:
Press = 138,30 kN
Pull = 137,60 kN.
Cyclic Lateral Press Load on the 3rd Cycle=130.7kN. Cyclic Lateral Pull Load on the 3rd Cycle=131.0 kN.
So that the load carried by the specimen is able to: Press Load = 130.70/138.30 (100%) = 94.50% > 75% (OK)
Pull Load = 131.00 / 137.60 (100%) = 95.20%> 75% (OK).
The maximum load at Story Drift : 3,50 % Cycle 3:

3) Energy dissipating Relative ratio (β) is comparing between broad Hysteretic loop formed by the vast parallelogram formed by the intersection at end Hysteritic Loop on Story Drift with the Story Drift stiffness
   a. Extensive formation of Energy by Hysteretic Loop Story Drift Ratio at 4.50%
      Energy dissipation of 4.50% Story Drift Cycle 3rd=16,143,803Nmm
      Area of Parallelogram formed by the end of intersection Loop Hysteritic: (E1 + E2) + (θ'1+ θ'2)=19,080,808.6Nmm
      ComparisonValue=16143803/19080808,6=0.85
      0.85 > 0.125 ... (OK).
   b. Area of the formation Energy by Hysteretic Loop at Story Drift Ratio 3.50% :
      Energy Dissipation 3rd cycle = 10,757,962 Nmm
      Area of Parallelogram formed by the intersection of Loop Hysteritic end: (E1 + E2) + (θ '1+θ '2) = 14759808,4,6Nmm
      Comparison Value=10757962/14759808,4 = 0.73
      0.73 > 0.125 ... (OK).

4) Value comparison The gradient Hysteritic Loop bordered by limits abscissa-X and + X on the X axis
   a. Comparison of Curva The gradient according to the requirements of ACI 374.1.05 3rd Cycle at Story Drift 4.50%
      Press :
      $$\frac{\text{Gradien Siklus ke}-3\text{ Story Drift }4{,}50}{\text{Gradien Siklus ke}-1\text{ Story Drift }0{,}002}=\frac{\tan 39{,}66}{\tan 82{,}77}=0{,}105$$
      0,105 > 0,05 … (OK)
      Pull :
      $$\frac{\text{Gradien Siklus ke}-3\text{ Story Drift }4{,}50}{\text{Gradien Siklus ke}-1\text{ Story Drift }0{,}002}=\frac{\tan 0{,}05}{\tan 82{,}12}=0{,}000121$$
      0,000121< 0,05 (not OK)
   b. Comparison of Curva The gradient according to the requirements of ACI 374.1.05 3rd Cycle at Story Drift 3.50%
      Press:
      $$\frac{\text{Gradien Siklus ke}-3\text{ Story Drift }3{,}50}{\text{Gradien Siklus ke}-1\text{ Story Drift }0{,}002}=\frac{\tan 44{,}79}{\tan 82{,}77}=0{,}126$$
      0,126 > 0,05 … (OK)
      Pull :
      $$\frac{\text{Gradien Siklus ke}-3\text{ Story Drift }3{,}50}{\text{Gradien Siklus ke}-1\text{ Story Drift }0{,}002}=\frac{\tan 41{,}24}{\tan 82{,}12}=0{,}121$$
      0,121 > 0,05 (OK)

V. CONCLUSION

Conclusion The results of Analyze beam-column joint of Specimen interior was as follows:

1) Strong Joint Slide.
   Results:
   Strong Slide Joint Press = 470.9 kN
   Strong Gesr Taik=465.8kN
   Planning Result=409kN
   Experimental Results > Results of Planning.
   Structure qualified
2) Yield Load capacity directly Initial Load.
   Ideal load(Pi) conditions on the elastic Story Drift 1,0 % the first cycle :
   P_i (press) = 108, 50 kN ; P_i (pull) = 106,20 kN
   Maximum load (Py) yield conditions on Story Drift 3,5% 13rd cycle :
   P_y(press) = 138,30 kN ; P_y (pull) = 137,60 kN.
   $P_y/P_i \geq 1{,}20$ 138,30/108,50 ≥ 1,20 (press) ⟶ 1,27 > 1,20 ………...(OK)
   $P_y/P_i \geq 1{,}20$ 137,60/106,20 ≥1,20 (pull) ⟶ 1,29 > 1,20 …………(OK), Structure qualified
3) Structural stability:
   a. The maximum load at 4.50% Story Drift :
      Load Press = 114.80/142.30 (100%) = 80.60% > 75%…(OK), Structure qualified
      Load Pull = 99.30 / 135.70 (100%) = 73.17% <75% (not OK), Structure not qualified.
   b. The maximum load at Story Drift : 3,50 % Cycle 3:
      Press Load = 130.70/138.30 (100%) = 94.50% > 75% (OK), Structure qualified.
      Pull Load = 131.00 / 137.60 (100%) = 95.20%> 75% (OK), Structure qualified.
   c. Energy dissipating Relative ratio (β).
      1. Extensive formation of Energy by Hysteretic Loop Story Drift Ratio at 4.50% :
         ComparisonValue=16143803/19080808,6 =0.85
         0.85 > 0.125 ... (OK), Structure qualified.
      2. Area of the formation Energy by Hysteretic Loop at Story Drift Ratio 3.50% :
         ComparisonValue=10757962/14759808,4 =0.73
         0.73 > 0.125 ... (OK).Structure qualified.
   d. Value comparison The gradient Hysteritic Loop bordered by limits abscissa-X and + X on the X axis:
      1. Comparison of Curva The gradient according to the requirements of ACI 374.1.05 3rd Cycle at Story Drift 4.50% :

Press :

$\frac{\text{Gradien Siklus ke}-3\text{ Story Drift }4{,}50}{\text{Gradien Siklus ke}-1\text{ Story Drift }0{,}002} = \frac{\tan 39{,}66}{\tan 82{,}77} = \quad 0{,}105$

0,105 > 0,05 … (OK), Structure qualified.

Pull :

$\frac{\text{Gradien Siklus ke}-3\text{ Story Drift }4{,}50}{\text{Gradien Siklus ke}-1\text{ Story Drift }0{,}002} = \frac{\tan 0{,}05}{\tan 82{,}12}$

$= \ 0{,}000121$

0,000121< 0,05 (not OK),

Structure not qualified.

2. Comparison of Curva The gradient according to the requirements of ACI 374.1.05 3rd Cycle at Story Drift 3.50% :

Press :

$\frac{\text{Gradien Siklus ke}-3\text{ Story Drift }3{,}50}{\text{Gradien Siklus ke}-1\text{ Story Drift }0{,}002} = \frac{\tan 44{,}79}{\tan 82{,}77} = 0{,}126$

0,126 > 0,05 … (OK), Structure qualified.

Pull :

$\frac{\text{Gradien Siklus ke}-3\text{ Story Drift }3{,}50}{\text{Gradien Siklus ke}-1\text{ Story Drift }0{,}002} = \frac{\tan 41{,}24}{\tan 82{,}12} = \ 0{,}121$

0,121 > 0,05 (OK), Structure qualified.

Resume**:**

In general, the model structure is eligible, although the structure stability test results are not eligible while receiving force Pull specimen on Story Drift 4.50%, but at 3.50% Story Drift has qualified. Similarly, in comparison Gradient, only on Dirft Ratio 4.50% who do not qualify. However, as is well known, both NEHRP regulations, ACI and SNI, all specify that if the Drif Ratio 3.50% has been met means that the structure has been designed to qualify. So this research, Strong Joint Shear and ductility qualified and acceptable.

Acknowledgement

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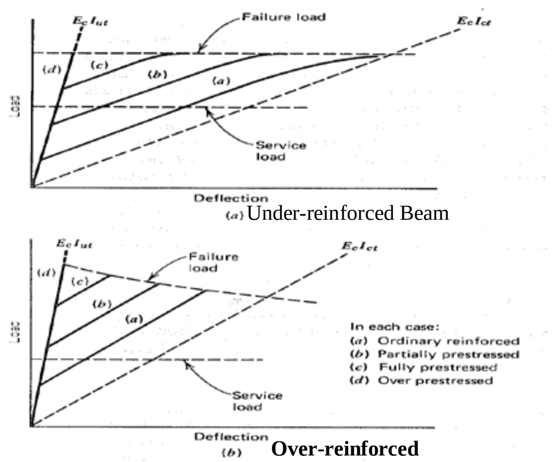

Figure 1. Idealization of the load-deflection curve for the beam with some variation of prestressing force. (Arthur H Nilson, 1987)

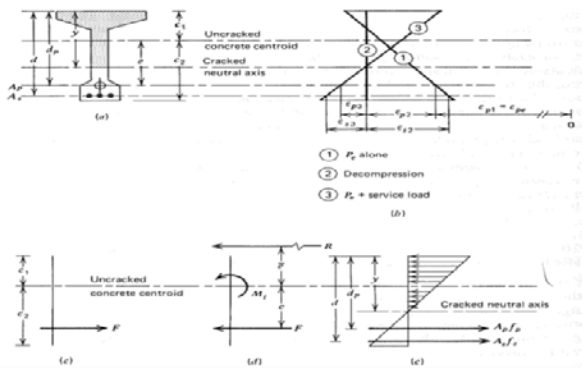

Figure 2. Basic cross-sectional analysis of cracks: (a) Cross-section cracks, (b) Strain of concrete and steel, (c) prestressed force, (d) force on the cross-crack, (e) Resultant stress (Arthur H Nilson, 1987)

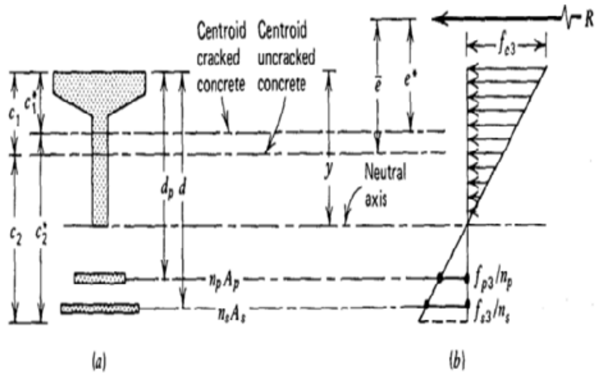

Figure 3. Transformation of partial prestressed cracked beam section.(a) the transformation of cracked section, (b) tension (Arthur H Nilson, 1987)

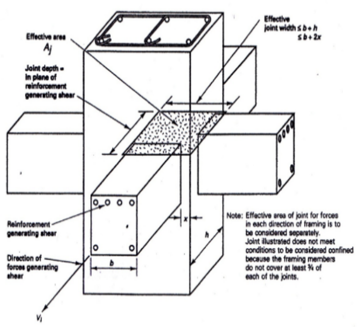

Figure 4. Broad efektive in the Joint (Aj) (Fanella David A, Munshi Javeed. A, 1998)

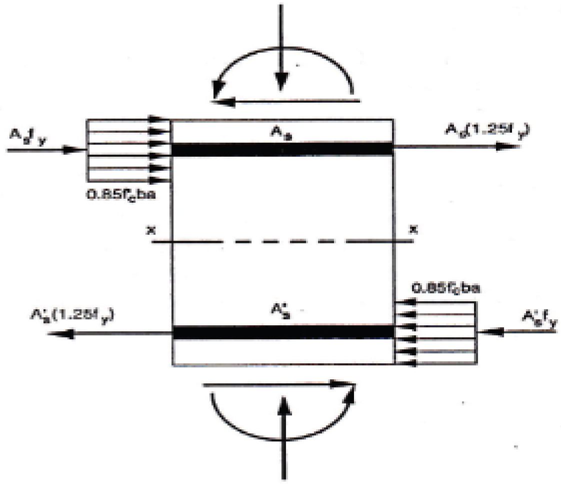

Figure 5. Horizontal shear force on the Beam-Column Joint (Fanella David A, Munshi Javeed. A, 1998)

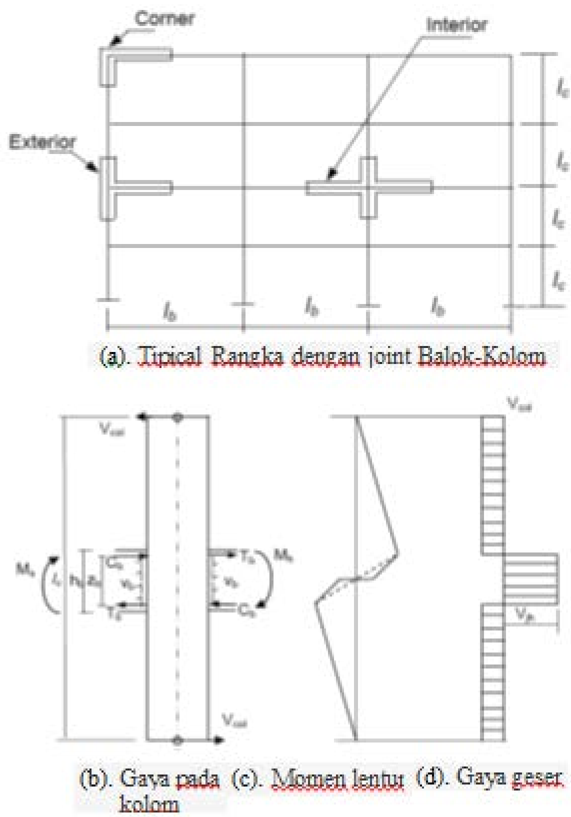

Figure 6. Horizontal shear force on interior joint (Uma & Meher Prasad, 2006)

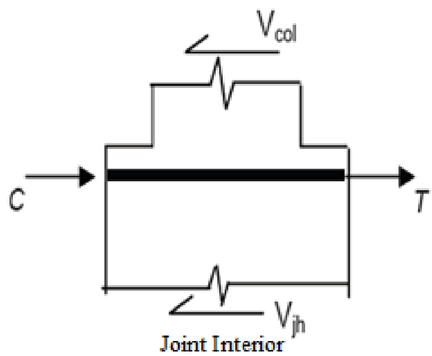

Figure 7. Balance on Joint shear (Uma & Meher Prasad, 2006)

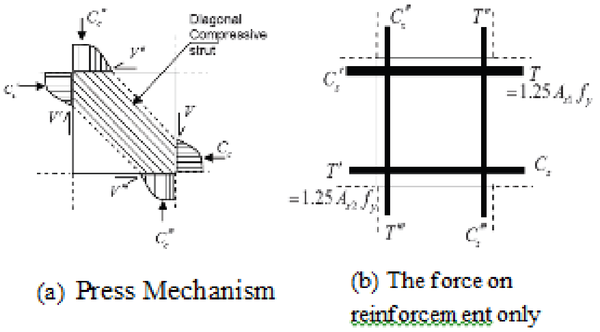

Figure 8. Idealization of the behavior of Beam-Column Joint (Uma & Meher Prasad, 2006)

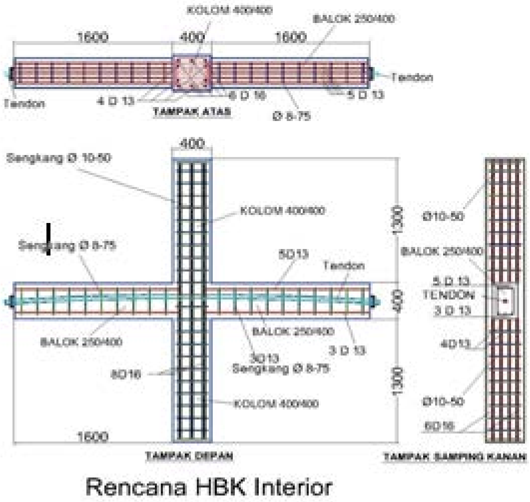

Figure 9. Design Interior Joint Spesimen

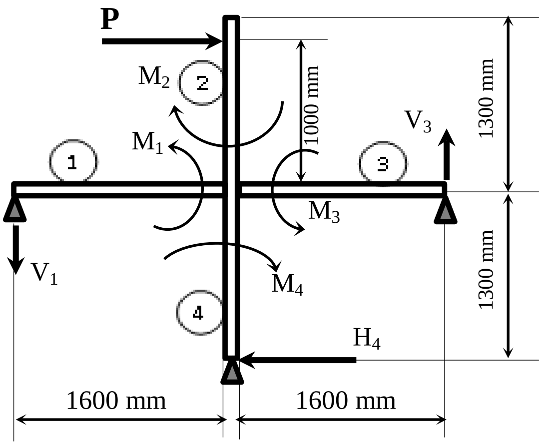

Figure 10. Moment distribution in the specimen Interior

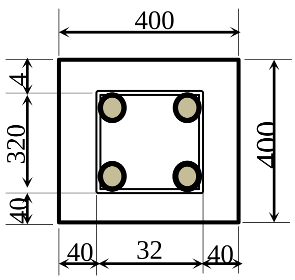

Figure 11. Stirrups position

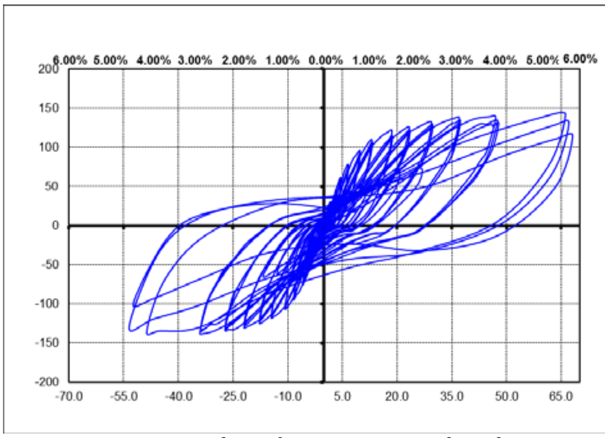

Figure 12. Relationships curve V Lateral Load and Lateral deflection at the top of column (tr-2)

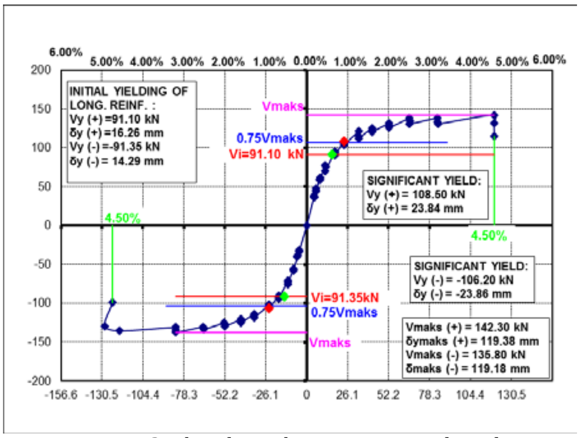

Figure 13. The relationship curve V Lateral Load and Lateral Deflection Peak Each Cycle at Peak Column (Tr-2)

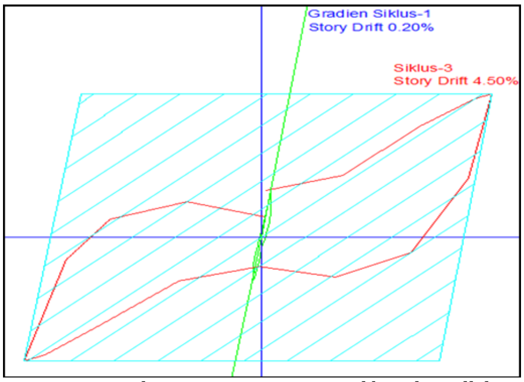

Figure 14. Energy dissipation curves area and broad Parallelogram in 3rd Cycle at the Story Drift 4:50% with gradient 0.200%

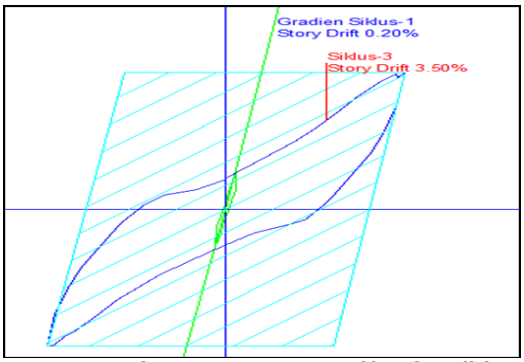

Figure 15. Energy dissipation curves area and broad Parallelogram in 3rd Cycle at the Story Drift 3:50% with gradient 0.200%

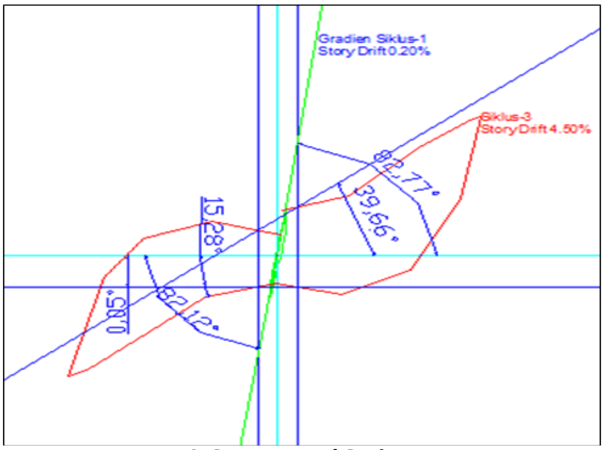

Figure 16. Comparison of Gradients curves in 3rd Cycle at Story Drift 4:50%

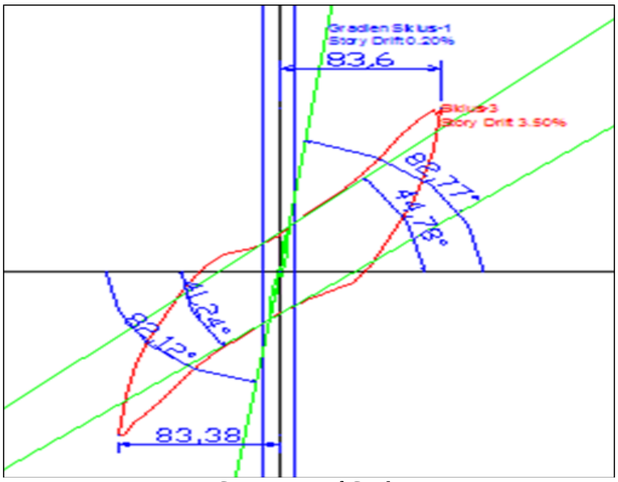

Figure 17. Comparison of Gradients curves in 3rd Cycle at Story Drift 5:50%

TABLE 1.
SPECIFICATIONS AND CRITERIA OF THE TEST SPECIMEN

Jenis Struktur	Element of Structure (cm)	Longitudinal Reinforcement	Stirrups	Tendons amount	Specimens amount
Interior Beam-Column Joint	Beam 25/40	Tensile Bar $5D_{13}$ / Compressive Bar $3\ D_{13}$	∅8 - 75	1 (2Strand)	1
	Column 40/40	$6D_{16} + 4D_{13}$	∅10 - 50	-	

TABLE 2.
THE LATERAL LOAD DATA AND DELECTION AT PEACK OF COLUMN.

No.	V Loads (kN)	δ (Tr_2) (mm)	Story Drift (%)	No.	V Loads (kN)	δ (Tr_2) (mm)
0	0	0	0,000	0	0	0,00
64	38,00	4,82	0,200	77	-33,10	-4,80
90	36,40	4,76	0,200	103	-32,40	-4,84
116	36,40	4,74	0,200	129	-32,10	-4,88
155	47,60	6,06	0,250	170	-38,30	-5,78
185	44,60	5,96	0,250	200	-40,30	-6,04
215	44,00	5,96	0,250	230	-40,00	-6,02
256	61,80	8,42	0,350	270	-58,20	-8,40
284	59,20	8,38	0,350	298	-55,90	-8,42
312	58,50	8,40	0,350	326	-55,60	-8,42
351	77,70	11,96	0,500	366	-75,40	-12,04
380	75,70	12,02	0,500	394	-72,80	-11,98
408	70,50	11,96	0,500	422	-70,50	-11,90
448	95,60	17,90	0,750	462	-94,60	-17,92
476	93,30	17,92	0,750	490	-92,00	-17,88
504	90,30	17,88	0,750	518	-90,60	-17,84
544	108,50	23,84	1,000	558	-106,20	-23,86
572	105,90	23,82	1,000	586	-103,20	-23,86
600	103,90	23,84	1,000	614	-101,90	-23,82
641	121,10	33,36	1,400	657	-118,10	-33,32
673	114,50	33,32	1,400	689	-115,50	-33,46
705	112,50	33,36	1,400	721	-113,50	-33,34
749	125,40	41,66	1,750	765	-124,70	-41,70
781	122,10	41,66	1,750	797	-121,80	-41,68
813	120,40	41,66	1,750	829	-120,40	-41,74
857	131,40	52,42	2,200	873	-130,00	-52,40
889	128,70	52,52	2,200	905	-127,10	-52,42
921	125,70	52,38	2,200	937	-125,40	-52,54
966	138,00	65,54	2,750	984	-133,70	-65,54
1002	134,00	65,74	2,750	1020	-131,70	-65,70
1038	130,70	65,52	2,750	1056	-130,40	-65,94
1086	138,30	83,34	3,500	1104	-137,60	-83,42
1122	133,70	83,42	3,500	1140	-134,30	-83,52
1158	130,70	83,60	3,500	1176	-131,00	-83,38
1201	142,30	119,38	4,500	1218	-135,70	-119,18
1217	131,40	119,38	4,500	1244	-129,40	-128,78
1233	114,80	119,58	4,500	1270	-99,30	-123,78
Maximum Loads	142,30	119,38	Maximum Load		-135,70	-119,18