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# Use of Steel Plate Mechanical Anchor for Strengthening Bamboo Reinforced Concrete Beam Column Joints

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A B S T R A C T
Bamboo can be employed as a substitute for steel in the longitudinal reinforcement of reinforced concrete. The object of this study was to investigate the influence of mechanical anchorage on the shear load capacity of bamboo reinforced concrete beam column joints. Four full scale beam column joint specimens with differing anchor lengths were tested experimentally in the Brawijaya University Civil Engineering Laboratory with cyclic loading. The mechanical anchorages (headed bars) consisting of the head and foot sections, the heads of 3 mm thick steel plate of 160 mm x 25 mm, two steel reinforcing rods of 6 mm diameter as the legs of 150 mm, 200 mm, and 250 mm, were welded to the heads. There were four headed bar lengths A1 15 cm, A220 cm, A3 25 cm, and B (the hooked anchor) 20 cm in length. The headed bars were installed on the joints and the specimens subjected to cyclic loading. The test results indicated that the maximum shear load was 9.24 kN for all specimens with headed bars (A1, A2, and A3) and8.10 kN for the conventional hooked specimen (B). The best seismic performance was for specimens A2 and A3.



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## INTRODUCTION

*Research Significance:*

Since the 1960’s, numerous experimental tests and analytical studies have been conducted to investigate the performance of reinforced concrete (RC) beam-column connections subjected to lateral earthquake loading. The problem is that many structurescollapse during earthquakes due to severe damage to beam column connections.

When a building is subjected to earthquake loads, it can collapse from beam column joint failure, which has been attributed to the lack of joint confinement. Based on the results of previous studies conducted by the authors and other research (Nindyawati and Umniati, 2009; Umniatiand Karyadi, 2009; Umniati and Nindyawati, 2010; Umniati and Nindyawati, 2011) on confinement model development of earthquake resistant bamboo reinforced concrete structures, indicated that the collapse of the structure was mainly due to damage to the beam column joints, while the outside of beam column joints were still in good condition with onlya few hair cracks in the area nearest the beam column joints. This was supported by the axial load test results on column specimens taken from bamboo reinforced concrete frames that had failed as a result of cyclic loads. These axial test results indicatedthat thecolumncould withstand an axial load of 12 tons - 17.5 tons. These results were well above the unconfined axial column capacity of 1.8 tons, and showed thatthe effect of confining on the column section was working properly and that the reinforcing bamboo was still well bondedto the concrete.For this reason, it is necessary to strengthen the core of beam column joints by using mechanical anchorages in order to better the overall performance of concrete structures. Headed reinforcing bars (referred to as “headed bars” hereafter) are becoming increasingly popular as longitudinal and transverse reinforcement for relatively large reinforced concrete structures that are exposed to extreme loads such as earthquakes or blasts. The use of headed bars often provides an adequate solution to steel congestion, particularly at beam-column joints.

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*Codes:*

Until recently, there were no building codes for bamboo reinforced concrete in Indonesia. Therefore, to design and analyze the structure we still use existing codes for 'steel' reinforced concrete structures. And the use of headed bars as a mechanical anchorage at beam column joint already exists in ACI Codes. Section 12.6 provisions of ACI 318-08 detail the development of headed and mechanically anchored deformed bars for the first time in the Code series. Prior to this, Joint ACI-ASCE Committee 352 published design recommendations for headed reinforcement used in reinforced concrete beam-column joints (ACI 352R-02). ACI 318-08 also introduces new provisions (Section 3.5.9) for obstructions or interruptions of the bar deformations, which should not extend more than 2db from the bearing face of the head. ASTM A970/A970M-07,14 "Standard Specification for Headed Steel Bars for Concrete Reinforcement," should also be satisfied by the requirements of Section 3.5.9. (Kang*et al*, 2010).

For the design of headed bars in beam-column joints, ACI 352R-02, "Recommendations for Design of Beam-Column Connections in Monolithic Reinforced Concrete Structures," can be used, where the development length is defined as functions of (*fydb*/√fc') for Type 1 and Type 2 beam-column connections and the nominal shear strength of the joint is Vn= 0.083γ(√fc')bjhc, should be satisfied ØVn≥Vu where bj is the effective joint width and hc is the depth of the column in the direction of joint shear being considered. Ø=0.85 and Vu is design shear force in joint. The constant γ is given in Table 1 ACI R352-02.

*Failure Modes:*

A major concern in the seismic design of RC structures is the ability of members to develop their flexural strength before failing in shear. This is especially true for members framing at a beam column joint (beams and columns), where it is important to develop their flexural strengths before joint shear failure (Tsonos, 2007).

According to Kang, *et al*(2010)failure modes can be categorized into three different groups as follows: Category I: member flexural hinging followed by modest joint deterioration; Category II: member flexural hinging followed by joint failure; and Category III: joint failure prior to member flexural hinging.

Category I specimens are considered to exhibit "satisfactory seismic joint performance," while specimens in the other two categories exhibit "unsatisfactory seismic joint performance." The performance indexes include: 1) the ratio of measured peak moment to nominal moment capacity (*Mp*/*Mn*); 2) drift ratio at the point of 20% drop from the peak lateral load (δ0.8*peak*); 3) ratio of strain in the headed bar at the joint-member interface to yield strain; and 4) joint shear distortion during approximately 3.0% drift cycles, where *Mn*is estimated following ACI 318-08 procedures. Joint failure was assumed to occur prior to flexural hinging (Category III) if the ratio of (*Mp*/*Mn*) was less than 1.0 and no bar yielding was monitored by strain gauges.

## MATERIALS AND METHODS

*Description of Test Specimens And Material Properties:*

Several species of bamboo have been used by the authors as bars in concrete, such as 'petung' (*Dendrocalamusasper),* 'ori' (*Bambusablumeana*) and 'tali' (*Gigantochloaapus*). These 3 types of bamboo have proved to be extremely suitable for replacing steel bars in concrete (Umniatiand Nindyawati, 2010 & 2011). For this study, the bamboo was prepared as follows: 1) the bamboo was cut and split into the sizes required and then dried for about 28 days ; 2) when dry, it was painted with 2 coats of paint used for wood as water proofing with an interval 24 hours between coats ; 3) immediately after the second coat, the freshly painted bamboo was buried in sand. Sand adheres to the bamboo and improves the binding of the concrete to the bamboo and, as soon as the paint is dry, the bamboo is ready for use as bars in concrete (Kankam*et al*, 1988).

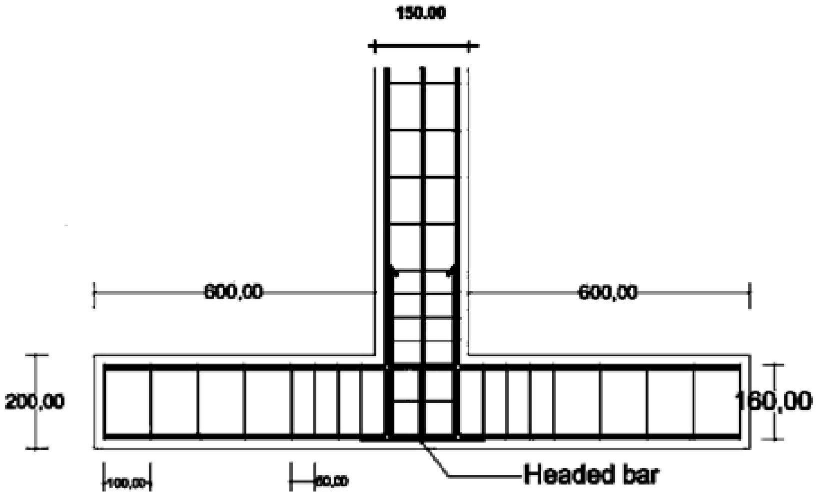
Four full scale roof interior beam-column specimens were designed and constructed for this experimental investigation. Reinforcement details of the specimens are shown in Fig. 1 had the same general and cross-sectional dimensions. They differ only on the type and leg length of the headed bars used.Specimens A1, A2, A3 and B had the same longitudinal column reinforcement, eight bamboo bars *(Bambusablumeana species)* of 10mm x 15 mm, while the longitudinal beam reinforcement of A1, A2, A3, and B consisted two bars on the top and three bars on the bottom of 10mm x 15 mm.

The leg lengths for the headed bar specimens A1, A2, and A3, were 150mm, 200mm and 250mm respectively. And the concrete 28-day compressive strength for all the specimens was 27.8 MPa. The average reinforcement yield strengths were as follows: for bamboo bars, 89.764 MPa and for the steel in the confinement, 424 MPa. An electrical-resistance strain gauge was bonded onto the headed bars of each specimen in the study.

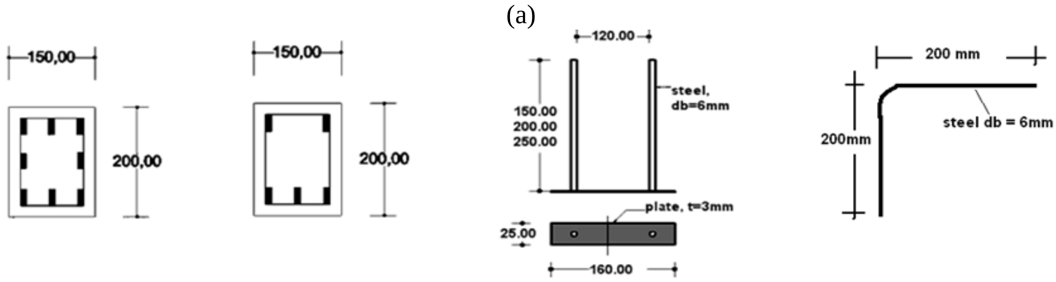
*Experimental Set-up And Loading Sequence:*

The general arrangement of the experimental set-up is shown in Fig. 2(a). All specimens were subjected to around 4 load cycles applied by slowly applying lateral force to the column's free ends, see the load history in

Fig. 2(c). The settings for the load peaks in the load history were 3x1.32kN, 6x1.32kN, 9x1.32kN, and12x1.32kN. No axial load was applied to the column or beams of the specimens.

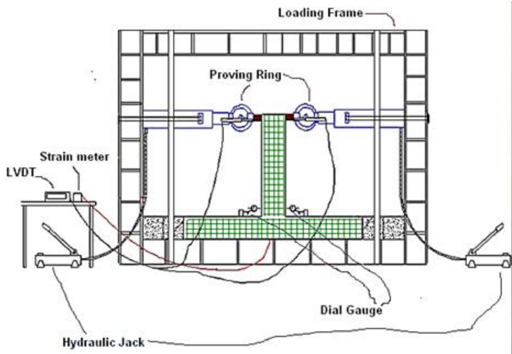


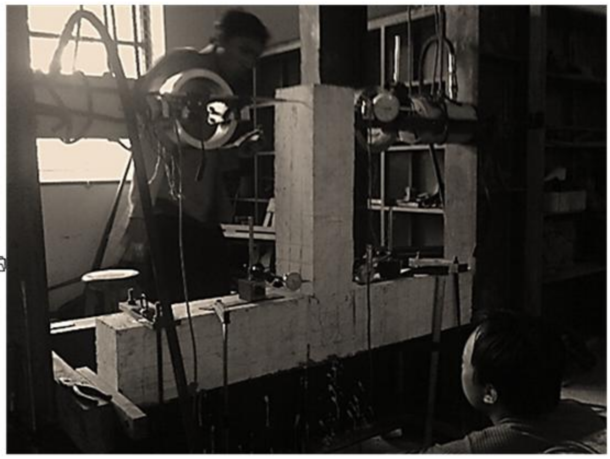

(a)



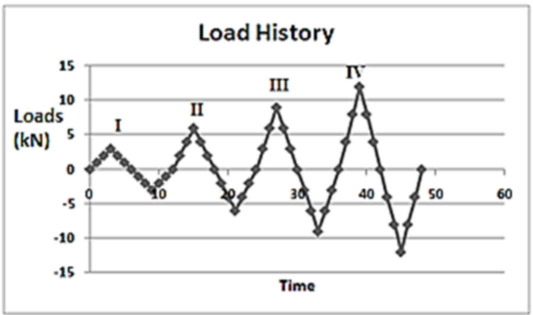

(b)

**Fig. 1:** (a) Specimens A1,A2,A3, and B; (b) Details of Column Section, Beam Section, Headed Bar (End Plate Bar) and Hooked Bar.



(a) (b)




(c)

**Fig. 2:** (a) General arrangement of the experimental set-up; (b) photograph of the test set-up; and (c) load history.

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*Results:*

*Load-Lateral Displacement Relationship:*

Plots for applied lateral load versus lateral displacement for all the specimens (A1, A2, A3, and B) are shown in Figure 3. Since the high of columns hc was 650 mm, maximum drift ratio reached was A1 4.13%, A2 5.15%, A3 6.03% and B 7.94%. Then this results summarized in table 1.

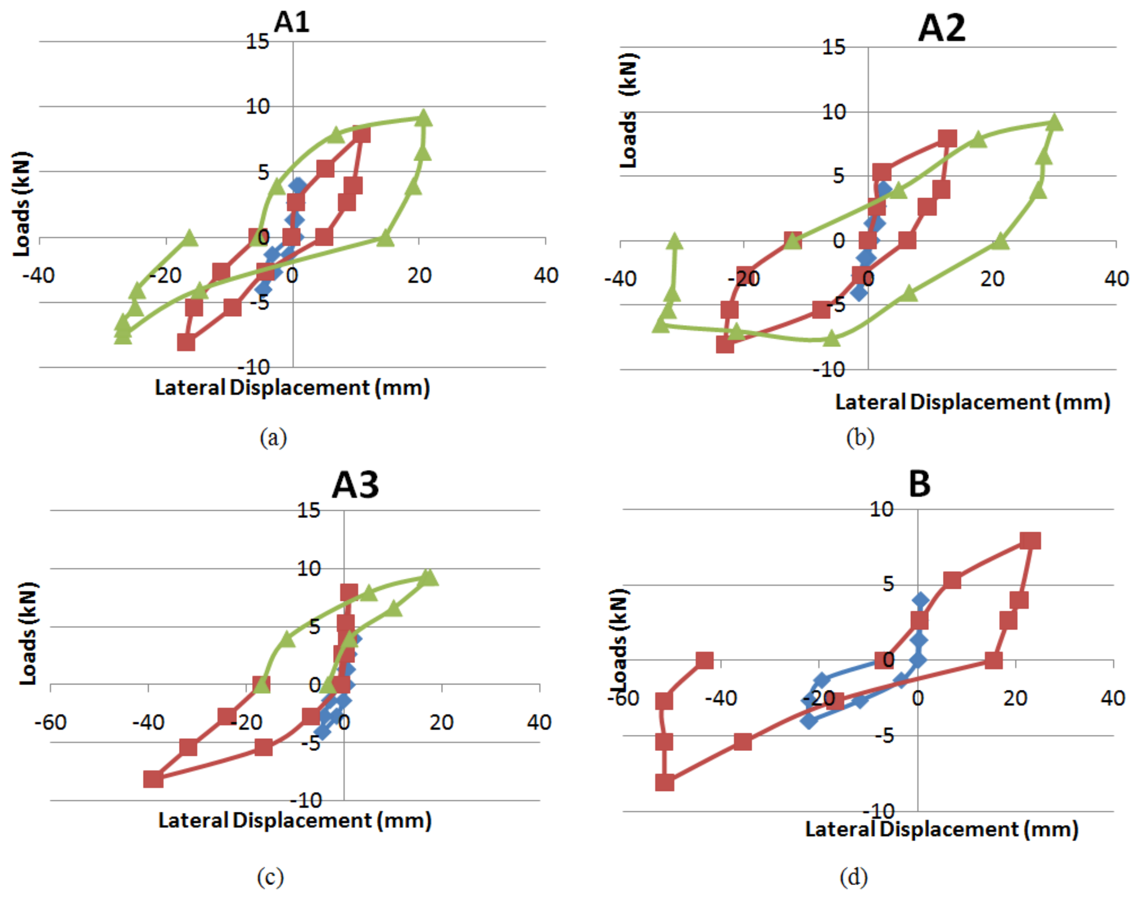


**Fig. 3:** Hysteresis loops P-Lateral Displacement for specimens A1, A2, A3, and B respectively.

**Table 1:** Maximum lateral load capacity, maximum lateral displacement, Drift ratio, and Hysteretic Energy Dissipation.

| Specimens | Maximum Lateral Load (kN) | Maximum Lateral Displacement (mm) | Maximum Drift Ratio (%) | Total Hysteretic Energy Dissipation (kNmm) |
|---|---|---|---|---|
| A1 | 9.24 | 26.841 | 4.13 | 359.773 |
| A2 | 9.24 | 33.466 | 5.15 | 695.851 |
| A3 | 9.24 | 39.195 | 6.03 | 224.906 |
| B | 8.10 | 51.669 | 7.95 | 393.390 |

After reaching the peak load in the last cycle load, the tests were stopped because of the serious damage such as rupture of reinforcing bamboo in the specimen. Therefore, lateral displacement at the point of 20% drop from the peak lateral load could not be obtained. From the hysteretic curves of load displacement relationship, total hysteretic energy dissipated by the beam column joints can be obtained by summing the area of curves for each specimen, namely, 359.773 kNmm, 695.851 kNmm, 224.906 kNmm, and 393.390 kNmm for A1, A2, A3, and B respectively.

*Load-Strain Curve:*

The material testing obtained steel headed bar yield strains of $3600x10^{-6}$. Figure 5 above indicated that none of the three specimens reached its yield strain. This meant headed bars in the joint core was still in the elastic range.

*Crack Patterns:*

Diagonal cracks in the joint region occurred in specimen B, while in specimen A1 hairline cracks appeared in its joint region, as shown in Fig.6 (d) and Fig. 6 (a) respectively.

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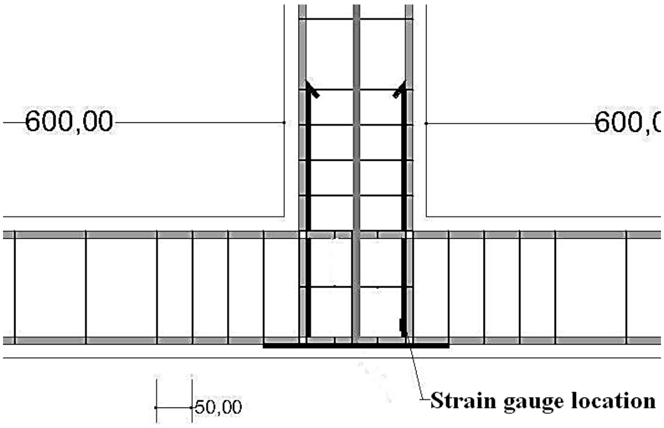



**Fig. 4:** Strain gauge location.

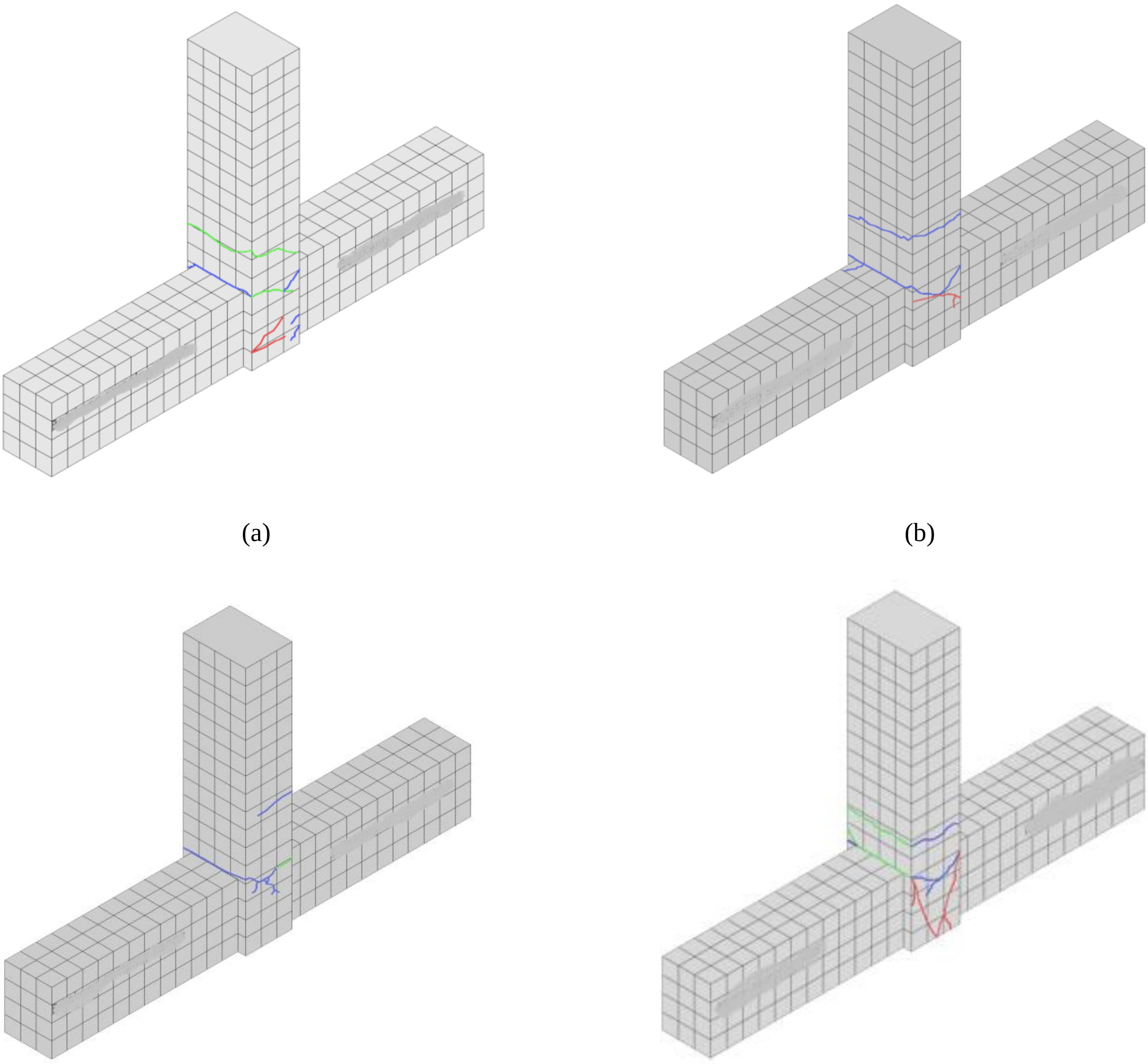



**Fig. 6:** Crack patterns of collapsed specimens A1 (a); A2 (b); A3 (c) and B (d).

Specimens A2 and A3, as shown in Fig.6 (b) and Fig. 6 (c) respectively, showed a typical flexural failure of the column with limited joint deterioration.

*Failure mode in specimens A1, A2, A3, and B:*

The failure mode in specimens A2 and A3, as expected, involved the formation of a plastic hinge in the column at the beam face. This formation of plastic hinges caused severe cracking of the concrete near the fixed column end in each specimen. For the specimen A1, cracks also appeared in the joint region but not as many as those in specimen B. Also, diagonal cracks occurred at the specimen B joint region which indicated the presence of joint failure.

*Discussion:*

Table 1 indicates that the maximum lateral loads measured for specimens A1, A2, and A3 are greater than those of specimen B. That is due to the contribution of steel plate anchor used in the joints area improving the capacity of the joints. However, if the results are compared to the calculated maximum lateral loads (ACI352R-02), specimens A1, A2, and A3 give 55.58 % and B gives 48.73 % lower than the calculated maximum lateral loads. This probably was due to the large standard deviation in the tensile strength of bamboo. And needs to be investigated more deeply.

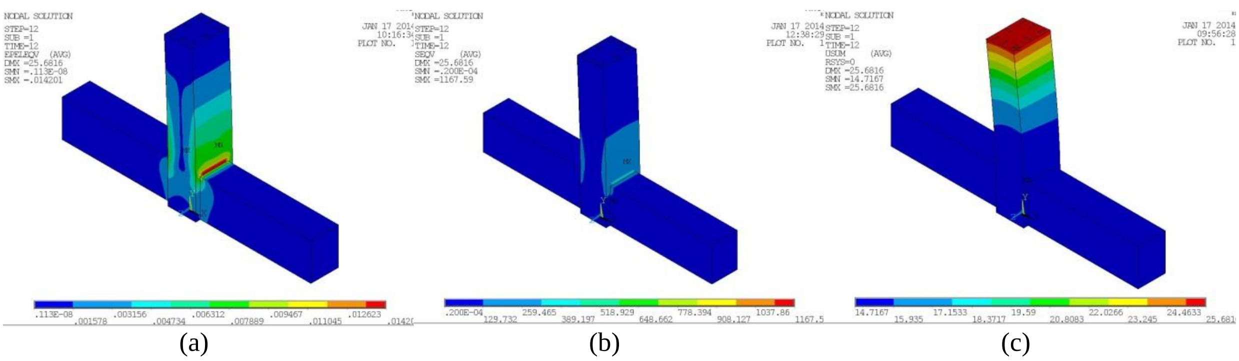
The load-strain relationship curves shows that the headed bars in all specimens (A1, A2, and A3) were still elastic, meaning that the joint areas were also still elastic in the end of the test. However, although the joint areaswere still elastic, the column bamboo bars in critical sections ruptured, and therefore the tests were stopped. It can be concluded that the type of failure of the joints A1, A2, and A3 areflexure in nature, as in Category I specimens(Kang*et al*, 2010).

The failure mode in specimens A2 and A3, as expected, involved the formation of plastic hinges in the columns at the beam faces. This caused severe cracking of the concrete near the fixed column ends in each specimen. For specimen A1, cracks also appeared in the joint region but not as many as those in specimen B. Also, diagonal cracks occurred in the specimen B's joint region which indicated the presence of joint failure, asCategory III specimens(Kang *et al*, 2010).

By summing the load-displacement hysteretic curves area of each specimen, the total hysteretic energy dissipation (E) of the beam column joints can be obtained, namely, 359.773 kNmm, 695.851kNmm, and 224.906kNmm, for A1, A2, and A3, respectively, and 393.390kNmm for B, see Table 1. This energy is but one part of the total energy demand. The complete total energy balance is given by (Symans *et al*, 2008) $E_1= E_s + E_k + E_D + E_H$ where, at a given instant in time, t, $E_1$= cumulative input energy; $E_s$= instantaneous strain energy stored by the structure; $E_k$= instantaneous kinetic energy of the moving mass; $E_D$= cumulative viscous damping energy; $E_H$= cumulative hysteretic energy dissipation. At the end of the earthquake ($t$=$t_f$), the kinetic energy is zero, the strain energy is zero for an elastic system (and zero or near zero for an inelastic system), and the cumulative hysteretic energy is equal to the energy demand (i.e., $E_{htf}$=$E_{\mathrm{Demand}}$). Then the magnitude of the earthquake resisted by the joint, can be obtained using equation Log E = 5.8 + 2.4 mB, for deep or intermediate events, where mB denotes long period body wave magnitude and E denotes thetotal energy balance in ergs (note: 1 Erg = 1.0168 x $10^{-7}$kNmm) or using equation Log E = 11.8 + 1.5 Ms for shallow event of earthquake, where Ms denotes surface wave magnitude(Gutenberg & Richter, 1956).

*Verification of Finite Element Model:*

Numerical analysis was conducted in The SPRS Laboratory, The Mechanical Engineering Departement, Brawijaya University, using ANSYS 14.5 software. The concrete was modeled as solid45, steel (shear reinforcement and headed bar) as shell63, and bamboo (longitudinal reinforcement) as beam4.



(a) (b) (c)

**Fig. 7:** Simulation Results: a) Strain distribution, b) Stress distribution and c) Displacement contour (units, stress: MPadisplacements: mm).

The simulation resultscompare to the experiment resultsshowed in table 2:

**Table 2:**.Comparison Result between Experimental and Numerical Study.

| | | Experiment | Ansys 14.5 (Academic version) |
|---|---|---|---|
| Strain in Headed Bar | A1 | 0.000929 | 0.000785 |
| | A2 | 0.000638 | 0.000467 |
| | A3 | 0.001986 | 0.000960 |
| Displacement (mm) | A1 | 26.841 | 25.682 |
| | A2 | 33.466 | 25.536 |
| | A3 | 39.195 | 22.011 |

Table 2 shows that in the numerical results the specimen behavior was the same as for the experimental results. Strain and displacement were lower than that of experimental results for the same load level.These were due to the homogeneity of the bamboo, and notas assumed due to the numerical analysis. In fact, as a natural resource, bamboo is not homogenous. Its tensile strength have larger standard deviations than steel. But for simplification in numerical analysis, it was assumed to be homogenous.

*Conclusion:*

The use of headed bars for beam column joints can improve structural performance. The length of the headed bar legs did not significantly influence the beam column joint load capacity. Three specimens using headed bars in differing leg lengths achieved the same maximum lateral load with almost the same displacement. But the longer the leg length of the headed bars, the greater their ability to absorb both tensile and compressive force so as to reduce cracking in reinforced concrete.

## ACKNOWLEDGEMENTS

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