EXPERIMENTAL STUDY ON THE FAILURE AND DEFORMATION OF BEAM-COLUMN JOINTS UNDER SEISMIC LOADS

*Nguyen Le Ninh[1], Vo Manh Tung[2]**

Abstract: *Beam-column joints play an important role, affecting the earthquake response of reinforced concrete (RC) frames. Their behavior is influenced by the way in which they damage the beams, columns and the complex interaction between the shear, bonding and confined mechanisms in a limited zone. To clarify the force-bearing and deformation of the joints that are being designed under earthquake loading and will be available in Vietnam, experimental studies are conducted in 1:1 scale specimens. This paper introduces the results of experimental research on the failure and deformation of the joints, as well as the causes of these behaviors.*

Keywords: *Beam-column joint, seismic experiments, earthquake, reinforced concrete (RC).*

Received: October 10th, 2017; revised: October 30th,2017; accepted: November 2nd, 2017

1. Introduction

The behavior of reinforced concrete (RC) moment resisting frame structures in recent earthquakes all over the world has highlighted the consequences of poor performance of beam-column (BC) joints. The BC joints are crucial zones for the transfer of loads effectively between the connecting elements (i.e. beams and columns) in the structures [1-4]. Shear failure of reinforced concrete BC joints has been recognized as an undesirable mode of failure, which lowers the seismic resistance of RC moment frame structures. The large amount of research carried out to understand the complex mechanisms and safe behavior of BC joints has gone into code recommendations. But it is worth mentioning that the relevant research outcomes on BC joints from different countries have led to conflicts in certain aspects of design [5,6]. The seismic provisions in major concrete codes, such as ACI, EC8, NZS, TCVN 9386:2012 .., the shear failure of BC joints is prevented. If the frame is designed to have weak beam-strong column mechanism, then the lateral strength of the frame could be calculated based on the flexural strength of beam sections. To achieve this goal, capacity design does not allow the premature joint shear failure before the beam yielding.

In this circumstance, the theoretical and experimental studies on the strength and ductility of the BC joints of RC frame under seismic loading are necessary to be carried out. The obtained results will be used to find out the failure modes, the ductility, the loading capacity, the stiffness as well as the energy dissipation of the seismic resistance joints based on the previous and modern design. Besides, the factors that influence the mentioned properties of the RC joints are given.

In the followings, the experimental studies on the failure and deformation of several RC joints, implemented in the IBST Seismic Lab, are going to be presented.

2. Details of Specimens

To investigate the joint performance (strength, deformation, ductility and energy dissipation), a series of three plane-frame interior BC joint test units are designed. The full-scale specimens are extracted from a frame of a 3-storey RC building. The building is designed with respect to seismic codes TCVN 9386: 2012 [7] and SP 14.13330.2014 [8]. The BC Joint test specimens are named as below:

- Specimen NK1: designed according to TCVN 9386:2012 [7].

- Specimen NK2: calculated according to the TCVN 9386:2012 (seismic actions only) and detailed according to the TCVN 5574:2012 [9].

[1] Asso.Prof.Dr, Faculty of Building and Industrial Construction. National University of Civil Engineering.
[2] Msc, Faculty of Building and Industrial Construction. National University of Civil Engineering.
** Corresponding author. E-mail: vo_manhtung@yahoo.com.vn.*

- Specimen NK3: designed according to the SP 14.13330.2014 [8].

All test specimens have same sizes (cross sections and lengths). Dimensions and details of specimens are described in Fig. 1 and Table 1. Material properties of the specimens are given in Table 2 and 3. The test specimens were cast in The Laboratory of Construction Testing and Inspection-National University of Civil Engineering.

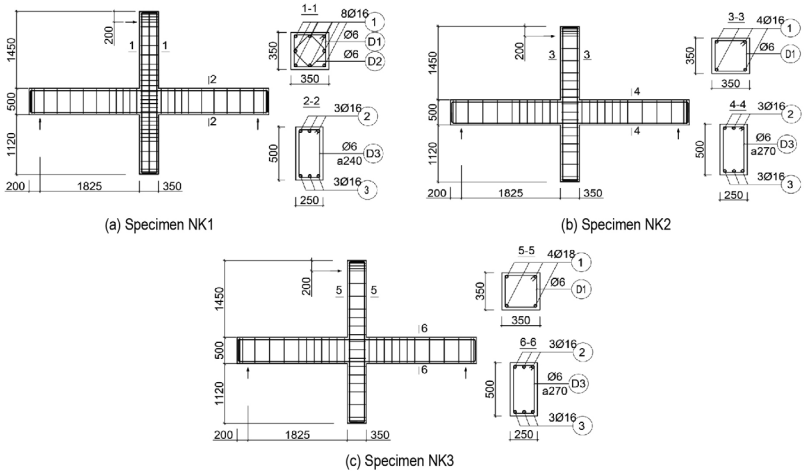


Figure 1. *Details of specimens*

3. Loading procedure and instrumentation

Figs. 2a and b show the overall test set-up. The test specimens were supported in a vertical position with a pin at the bottom of the column and free to move at the column top in horizontal direction either ways. An axial load of 300 kN was exerted on the top of the column with a heavy duty hydraulic jack. The both beam ends were supported by horizontal rollers.

A cyclic horizontal load is applied at the column top of the test specimens. The selected load is intended to cause forces that simulate high levels of inelastic deformations of the frames during a severe earthquake ground motion. The selected load routine shown in Fig. 3, consists of two stages. The first stage is load-controlled followed by displacement-controlled loading stage. In the load-controlled stage, two first cycle 1-2, test specimens were loaded in the lateral direction $V = 0.75V_i = 0.75 \times 59.23 = 45$ kN, where V_i: the theoretical load capacity of the specimen under lateral and vertical loads, determined from the their dimensions and material properties. The purposes of this load cycle were to determine the yielding displacement Δ_y and the actual stiffness K_{tn} of the test specimens. After the two initial elastic cycles, inelastic cycles

Table 1. *Technical parameters of specimens*

Specimens	NK1	NK2	NK3
Longitudinal reinforcement of beams A_s=A'$_s$ (Class AII)	3Φ16	3Φ16	3Φ16
Reinforcement ratio of beams ρ_d	0.52%	0.52%	0.52%
Reinforcement ratio of columns (Class AII)	8Φ16	4Φ16	4Φ18
The reinforcement ratio of columns ρ_c	1.48%	0.74%	0.94%
Beam stirrup $\rho_{đd}$ (Class AI)	Φ6a125; a240	Φ6a140; a270	Φ6a140; a270
Beam stirrup ratio	0.18%;0.09%	0.16%;0.08%	0.16%;0.08%
Column stirrup ρ_{dc} (Class AI)	Φ6a75, a177, Φ6a100, a187	Φ6a160, a240	Φ6a160, a240
Column stirrup ratio	0.37%;0.16% 0.28%;0.15%	0.1%;0.07%	0.1%;0.07%
Stirrup in the joint $\rho_{đn}$	6Φ6a75(D1); 6Φ6a75(D2)	3Φ6a160	3Φ6a160
Stirrup ratio in the joint	0.37%	0.1%	0.1%

Table 2. *Material properties of concrete and reinforcement*

Concrete				Reinforcement			
Specimen	**NK1**	**NK2**	**NK3**		**Φ18**	**Φ16**	**Φ6**
f_c at 28-day age (MPa)	30	29	31	f_y (MPa)	310	320	235
Age of Specimen at the tested time (day)	83	90	80	f_u (MPa)	480	510	400
f_c at the tested time (MPa)	31.5	32	31.7	E_s (MPa)	2.1×105		
ε_c	0.0025	0.0025	0.0025				
E_c (MPa)	30000	30000	30000				

were applied at progressively larger displacement ductility factors, with four full cycles being imposed on each level of ductility. The object of progressively increasing the ductility demand on the structure was to define as closely as possible the ductility demand at which the failure in the unit might occur, and to observe the behavior both before and after any such failure. The intermediate cycles to displacement ductility factor μ_Δ=0,75 were included on the basis that cyclic behavior under actual seismic attack would not consist merely of increasingly larger post-elastic cycles. The tests were to be continued since the specimens had lost 20% load-bearing capacity to find out the maximum plastic deformation and the main factor that caused the failure and the loss of the load-bearing capacity.

The instrumentation of the test specimens was designed to measure the overall response and the load resistance mechanism in the beam-column subassemblage. Three types of measuring device were used: load cell, LVDTs (Linear Variable Differential Transformers) and strain gauges. The test data were obtained at every load step in loading and unloading cycles and the reverse direction of loads, including lateral induced loads, displacements of column ends, shear deformation in the joint area, flexural and shear deformation of beams, columns and deformation of reinforcement in the critical zones of beams, column and joint... The locations of LVDT and strain gauges were displayed in Figs. 4 and 5.

4. The failure of specimens and the causes

4.1 The failure of specimens

4.1.1 Test specimen NK1

Specimen NK1 was designed according to the TCVN 9386:2012[7], that secured the rules of the ductile failure of the RC frame: the hinges were formed in beams first, the shear failure happened after the

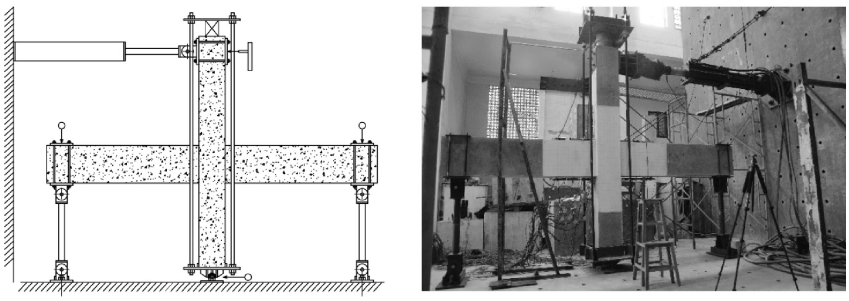


Figure 2. *Schematic of test set - up*

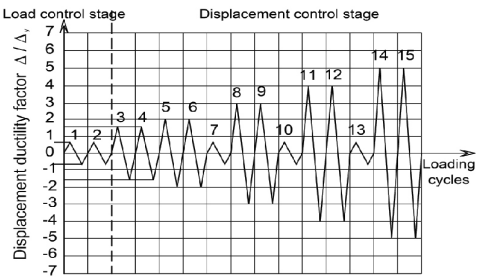


Figure 3. *Load routine*

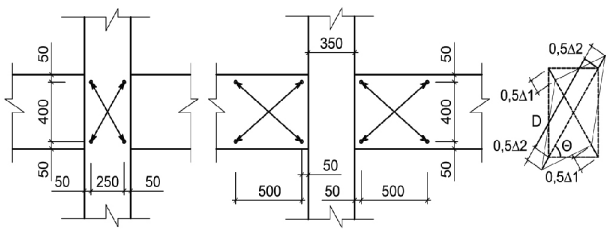


Figure 4. *LVDTs for joint shear deformation and for beams shear and flexural deformation*

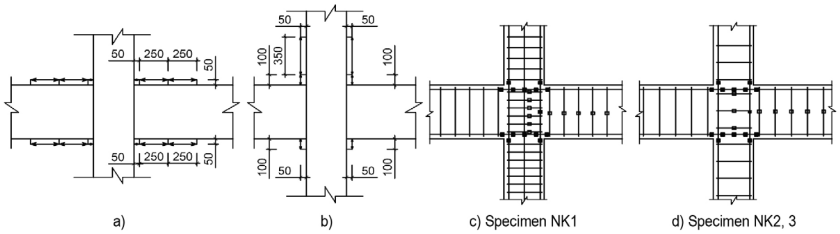


Figure 5. *The locations of LVDT to obtain the rotational displacement of beam and column ends And the position of strain gauges*

occurrence flexural failure and the joint was failed at last. With the design rules, the ratio of the yielding moment of the beam to that of the column of NK1: $M_{yi,c}/M_{yi,d}$ = 1.29. The final crack pattern is shown in Fig. 6 at the end of 19th loading cycle.

The images of the crack pattern show that:

- The beams were damaged with the cracks nearly or perpendicular to their axes. These cracks were uniformly distributed in a length of $2h_d$ (h_d=the height of the cross-section of the beams). When the load reversed, the cracks were connected to form continuous cracks running through the height of the beams.

- At the column areas around the joint, the cracks are also appearing. These flexural cracks nearly or perpendicular to the column axes were on both sides of the columns in the similar distance to the joint due to the reversal action of loading. In comparison with beams, the column cracks were smaller in both the number and the width and located on the length about $1.5h_c$ (h_c -the height of the cross-section of the column).

- At the joint, there was a net of inclined cracks parallel to the diagonal line of the panel joint. The distance between cracks was approximate equal. Under the effect of the reversal action of loading, the cracks divided the joint area as a canarium net.

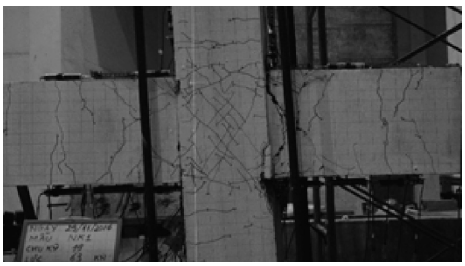


Figure 6. *NK1 specimen image at end of experiment - 19th cycle*

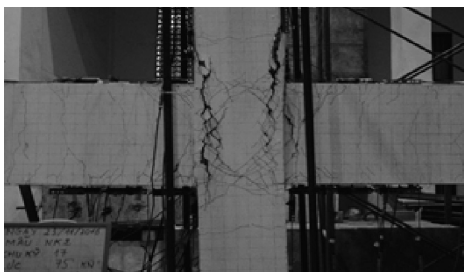


Figure 7. *Specimen image of NK2 at end of experiment - 17th cycle*



Figure 8. *NK3 specimen image at end of experiment - 14th cycle*

4.1.2 Test specimen NK2

Specimen NK2 was calculated according to TCVN 9386:2012 for seismic actions and detailed according the concrete design code TCVN 5574:2012 [9]. The moment ratio $M_{yi,c}/M_{yi,d}$ = 1.02. The stirrups in the joint were similar as those in the vicinity area of the columns with the reinforcement ratio ρ_{wn} = 0.1%. In this case, the risks of shear-flexural failure of column, beam and the joint were equal. The final crack pattern at the end of 17th loading period is shown in Fig. 6. The image of pattern cracks showed that the failure occurred in the joint region and its surrounding areas. This pattern cracks were very different with that at NK1:

- In comparison with NK1, flexural cracks of the beams were smaller in number and appeared in a shorter length (about $1.5h_d$). Most of the cracks were not long enough to connect to the crack on the opposite side to run through the whole section of beams.

- The flexural cracks of the upper and lower columns were small in number and their width were also very narrow. The cracks had not widened in the loading process.

- At four corners of the BC joint, there were many cracks perpendicular and parallel to the diagonal direction. The cracks intersected and formed diamond shaped panes adjacent to the beams' end. The cracks did not run into the central area of the joint. Thus, the considerable canarium shaped area in the joint seemed not to be cracked. The area inside the boundary of the joint was cracked, seriously to form two broken strips along with the longitudinal reinforcement of columns. The strips ran through the joint to the vicinity areas of the columns.

4.1.3 Test specimen NK3

Test specimen NK3 was designed and detailed according to the current Russian seimic design code SP14.13330-2014 [8]. The moment ratio $M_{yi,c}/M_{yi,d}$ was 1.12. The final crack pattern of this test specimen is shown in Fig. 8 at the end of 14th loading cycle. The crack pattern of NK3 and NK2 were quite similar to that of NK1, but the some differences are as follows:

- In the joint region of NK3, the inclined cracks in both directions developed in the central zone. Thus, the uncracked rhombic shaped area of the joint region of the NK3 was smaller than that of NK2. Although the inclined cracks were mainly formed in the vicinity areas of the beam ends, there was no splitting strips at the vertical sides of the joint region in NK3 that occurred in NK2.

- The cracks in the beams and columns around the joint region of NK3 are more numerous and wider than the NK2. At the two ends of the beam, many inclined cracks appear in the upper and lower beams as in the NK2.

4.2 The causes of the failure

The joint core is subjected to two types of actions that combined are generally known as joint shear [6]:

- Concrete flexural compression from beams and columns at the opposite corner of the joint (Fig. 9b);

The shear flow along its perimeter from beam and column bars by means of bond forces (Fig. 9c).

The resistance mechanism is composed by a compressed concrete diagonal strut roughly limited by the neutral axes of the end sections of the members (Fig. 9d) and by diagonal compression field-truss mechanism-consisting of horizontal hoops, intermediate column bars and inclined compressed concrete strips between shear cracks (Fig. 9e).

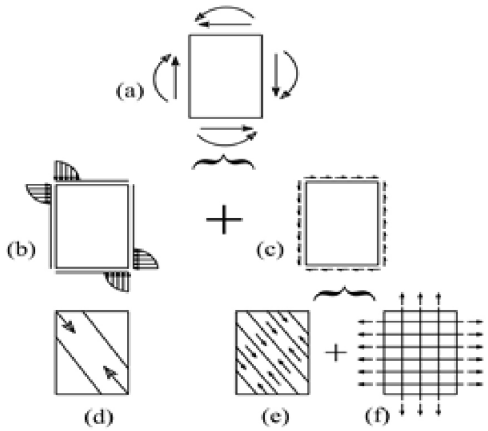


Figure 9. *The force transmission and bearing mechanisms in the joint*

Thus, failure of the joint is due to the inability of any of the "sub-mechanisms" depicted in Figs. 9d, e and f to carry successfully the load they are meant to sustain. It follows then that three different sources for joint failure can be pointed out: failure of the compressed diagonal strut, failure due to loss of bond resistance along the joint boundary and failure due to inability to develop a truss mechanism that can carry the diagonal tension by the premature yielding of the longitudinal bars intercepting the core.

The failures of the specimens NK2 and NK3 has not been documented in the specialist literature before, which is a combination of the three sources of the joint failure above. Based on the force transmission and bearing mechanisms in the joint region mentioned above, the cause of the failure of the joint is as follows:

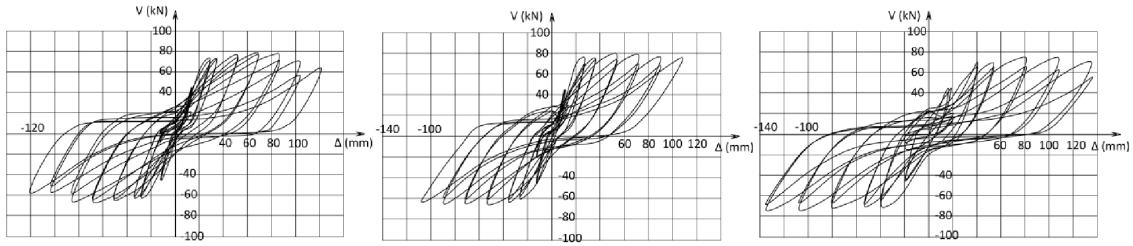
a) The mobilization of the truss mechanism is subject to the shear force depends on the bond of the reinforcement in the joint region. In order to prevent early damage to the bond under two-way effects, the concrete core confined is an extremely important factor. The confined effect is generated through the stirrup and intermediate reinforcement columns. The specimen NK1 has a much larger concrete confined effect than the other two. This is evidenced by the analysis of the rotational displacement data for the beams, columns and yield strain of the reinforcements of the three specimens.

The relationship of the storey shear V-lateral displacement Δ of the samples shown in Figs. 10a, b and *c* shows the hysteresis loop of all three specimens in a two-directional load of approximately symmetrically. All of the hysteresis loops exhibit a pinching to varying degrees in the specimens. The specimen NK1 has the pinching level smallest and the shortest, while the NK3 is the narrowest. This pinching effect begins to appear when the ductility $\mu\Delta$ is = 2 in NK1, while NK2 and NK3 appear earlier in the $\mu\Delta$ = 1.5. The cause of the pinching of the loops is the slip and the yield of the beam longitudinal bars. This indicates that the longitudinal bars of the beams on the NK2 and NK3 specimens lost bond earlier and had a larger slip.

- Deformation of the beams

Figs. 11a, b and c show the relationship between the bending moment and the rotation of the beam section at 50 mm from the right column face. These graphs show that, at the final load cycles before the end of the experiment, the rotational displacement of the beams of the NK1 specimen has a very large increase

(Fig 11a). The cause is the effect of concrete confined has made the longitudinal bar the beam through the joint of NK1 hold tight, no loss of bond and no slip so it can develop yield deformation, fully, while in NK2 and NK3, longitudinal bar can not develop full deformation due to premature adhesion loss and slip. The strain gauges on beam reinforcements show that in NK1 the longitudinal bar the beam starts to yield in the 3rd cycle at the column face, particularly in the near to center column is not yielding or yielding later in the 9th cycle (negative), 11th (positive), while in NK2 and NK3 specimens the longitudinal bar the beam begins to yield at the 3rd cycle at the column face, but at the near to column center is yielding soon at 3rd, 6th cycle of NK2 and 3rd, 5th cycle of NK3.



a) Specimen NK1 b) Specimen NK2 c) Specimen NK3

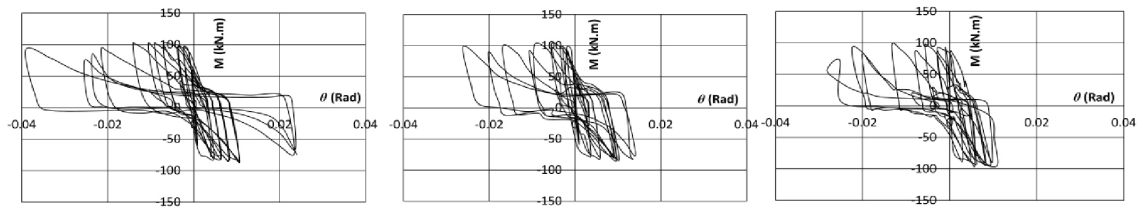
Figure 10. *The relationship of the storey shear V- lateral displacement Δ*

- Deformation of the columns.

The strain gauges mounted on the column reinforcement shows that the reinforcement on the right side of the NK1 column face starts to yield in the 8th cycle, particularly at the center of the beam of the 12th cycle, while the NK2 and NK3 begin to yield relatively early in 5th cycle, at the center beam is not yielding. This proves that the NK1 column reinforcement has developed a good plastic deformation, while in NK2 and NK3 specimens, only plastic deformation at the beam face sections of the column.

This behavior of the column reinforcement in the joint has been reflected in the curves in Fig. 12. In the NK1 specimen, the rotational displacement θ_c of the column is small and almost linearly stable until the column reinforcement at the horizontal cross-section of beam face beginning to yield at level $\mu\Delta = 3$ (8th cycle).

Then, the rotation increases nonlinearly but is much smaller than the other two. For the NK2 and NK3 specimens, until the column reinforcement at the horizontal cross-section of the beam face is yield at level $\mu\Delta = 2$ (5th cycle), the rotation of the column is the same as that of the specimen NK1, then nonlinear increase. Thus, column reinforcements in NK2 and NK3 can not develop the full plastic deformation as in NK1. This shows that the rotational displacement of the columns in the NK2 and NK3 specimens is not a complete plastic displacement as in the NK1 but rather a composite of yielding reinforcement, compression and tensile of concrete and reinforcement elongation is similar to the case of beam deformation.



a) Specimen NK1 b) Specimen NK2 c) Specimen NK3

Figure 11. *The relationship between the bending moment and the rotation of the beam section at 50 mm from the right column face*

Thus, it can be seen that the NK1 specimen is capable of taking form the truss mechanism while the NK2 and NK3 do not have this capability. The diagonal cracks of the NK2 and NK3 specimens cannot be formed, instead they are still compressed at corners due to the rotational displacement at the ends of the beams and columns (Figs. 7 and 8).

b) As the inelastic loading cycles increase, the role of the compression strut (Fig. 9d) decreases and increases the contribution of the truss mechanism (Fig. 9e) to the transmission of the horizontal shear force. Until the 4th cycle, the cracks images of all three specimens are very similar, indicating the same behavior that is the compressive behavior of the beams and columns on the joint at the first inelastic cycles, before

starting to mobilize the truss mechanism. As the load cycles increase in the inelastic region, the NK1 joint is stressed to a smaller degree, by the beams and then the column is deformed plastic bending, while the NK2 and NK3 joints are subject to increased local compression from the large rotates of beams and columns due to the loss bond of reinforcement and the weakened confined concrete that is not capable of transmitting the diagonal force into the core of the joint.

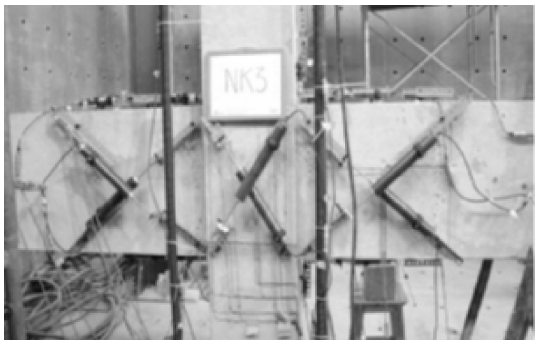


Figure 12. *Locations LVDT installed in diagonal*

c) The spalling of concrete along the column reinforcement on either side of the NK2 specimen (Fig. 7) is a combination of three effects: crack expansion at the contact section between beam and column due to the beam is rotated when the reinforced beam is yield and losses bond, the corners of the panel joint are compressed local by the column rotation and especially the beam around, buckling out due to the loss bond and the distance of stirrups too large not enough to create the effect confined of column reinforcement.

5. Shear deformation of joints

Shear deformation in the joint provides a measure of joint stiffness. The two LVDTs placed diagonally across the corners of the joint panel measured the elongation and shortening of the diagonals with load (see Figs. 4a and 12). The average shear deformation of the joint panel was computed by the equation:

$$\gamma_{tb} = \frac{\Delta_1 + \Delta_2}{D \sin 2\theta} \tag{1}$$

where Δ_1 and Δ_2 are the change in the lengths of the diagonals, D is the length of the diagonal (distance between LVDT ends) and θ is the angle between the diagonal and the horizontal member. The story shear V versus joint shear deformation γ for three specimens are shown in Figs. 13a, b and c. These figures show that the joint shear deformation of specimen NK1 is very low compared with that of specimens NK2 and NK3. Figs. 13b and c show that the joint deformations of the specimens NK2 and NK3 are very low in the first two loading cycles, but after that the excessive shear deformation and stiffness degradation in the joint are occurring.

The relationship between the joint shear deformation γ and the displacement ductility $\mu\Delta$ given in Fig. 13d, shows a very large variation in the magnitude of the joint shear deformation designed in different ways. The NK1 joint is designed according to a modern design standard, attaching importance to ensuring the stiffness, durability and ductility of the joint, other than NK2 and NK3, stirrup as in the adjacent columns only. The joint shear deformation of these two specimens was more nonlinear than the NK1. Particularly for NK2 and NK3 specimens, the only difference is the diameter of the longitudinal bars of column, which makes the ratio of ultimate bending strength of the column and beam in NK2 with $M_{yi,c}/M_{yi,d}$ = 1.02 and NK3 by $M_{yi,c}/M_{yi,d}$ = 1.12. Thus, the increase $M_{yi,c}/M_{yi,d}$ ratio in NK3 appears to be a condition that increases the joint shear deformation when not properly confined.

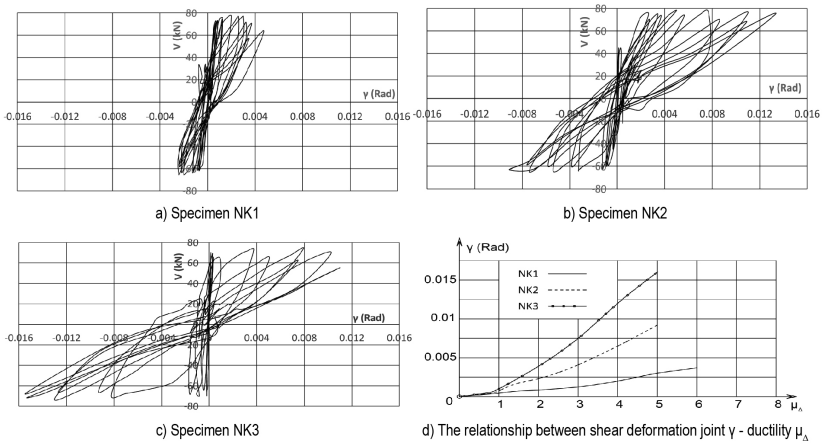


Figure 13. *Storey shear - Shear deformation joint, The relationship between shear deformation joint γ - ductility μ_Δ*

6. Conclusions and recommendations

Based on the analysis of the failure modes of the test specimens and the joint shear deformation, the following conclusions may be drawn:

- Different design methods lead to different behavior of the specimens. The NK1 pattern failure is ductile with hinges occurring in the beams near the column face. Damage to the joint and its deformation is relatively evenly distributed over the entire joint panel. The failure mode of NK2 and NK3 specimens is brittle. The joint panel of this specimen is crushed under the local compression, impact of the column and beam rotation, the beams and columns are not fully deformed plasticity and are not completely bending. The risk of failure (bending and shear) between the beams, columns and joints is nearly equal.

- Under the shear and bending of beams and columns, joints will be significantly deformed, even when designed in accordance to the modern seismic code (TCVN 9386:2012). Therefore, considering the deformation of the BC joint in the seismic design of moment resisting reinforced concrete frame system is very necessary.

- The experiment shows a tremendous importance of concrete confinement as recommended by most international codes. To prevent shear failure of the BC joints, both horizontal and vertical reinforcement are required. Confinement may be achieved with reinforcement that can exert clamping action across splitting cracks. Therefore, intermediate column bars, apart from their role as vertical joint shear reinforcement, are very important to prevent premature bond failure in the joint.

- Experiment shows that frame structures are designed in accordance to the Russian seismic standard SP 14.13330.2014 and Vietnam concrete code TCVN 5574:2012 are not suitable for developing a ductile failure mechanism in strong ground motions.

Acknowledgement

The authors would like to express grateful thanks to the colleagues in the Laboratory of Construction Testing and Inspection (NUCE) and the Seismic Laboratory (IBST) for their help.

References

1. Beckingsale C.W., (1980), *Post-Elastic Behavior of Reinforced Concrete Beam-Column Joints*, Research Report 80-20, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand.

2. Ninh N.L. (2007), *Earthquake and earthquake resistant structure design*, Construction Publishing House (in Vietnamese).

3. Paulay T., Priestley M.J.N., (1992), *Seismic design of reinforced concrete and masonry buildings*, John Wiley, USA.

4. Sangjoon P., Khalid M.M., (2012), *Experimental and Analytical Studies on Reinforced Concrete Buildings with Seismically Vulnerable Beam-Column Joints*, PEER Report 2012/03, Pacific Earthquake Engineering Research Center, USA.

5. Kim J., LaFave J.M., (2009), *Joint Shear Behavior of Reinforced Concrete Beam-Column Connections subjected to Seismic Lateral Loading*, Report No. NSEL-020, Department of Civil and Environmental Engineering University of Illinois, USA.

6. Mitra N., (2007), *An analytical study of reinforced concrete beam-column joint behavior under seismic loading*. University of Washington, USA.

7. TCVN 9386:2012, *Design of structure for earthquake resistance-Vietnamese standard*, Construction Publishing House (in Vietnamese).

8. SP 14.13330.2011 (2011), *Buildings in seismic zone-Russian standard*, Moscow, (in Russian).

9. TCVN 5574:2012, *Structural Concrete and Reinforced Concrete-Vietnamese standard*, Construction Publishing House, Hanoi (in Vietnamese).