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Experimental Investigation of the effect of cold joint on strength and stiffness degradation in RC beam-column connections

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Abstract — *in this paper the effect of cold joint has been discussed for two RC Beam-Column Connections. Specimen 1 was poured monolithically whereas, construction joint in specimen 2 at bottom and top column was provided according to section 3.2.2.2, ACI 224.3R-95. Both specimens were tested through quasi static cyclic loading under displacement controlled condition, and the effect of cold joint was investigated. It was concluded that due to construction joint in connections the maximum load carrying capacity decreases up to 39%. In addition, the strength degradation was more in specimen having construction joint in column. Similarly the stiffness degradation decreases up to 50% due construction joint. It was also observed that specimen having construction joint shows overall strength lost at 1.5% while specimen having no construction joint was maintain their strength up to 3% drift.*

Keywords*- Cold joint; beam-column connection; strength degradation; stiffness degradation*

I. INTRODUCTION

Construction joint is not avoidable due to several reasons in concrete structures. For example (a) continuous pouring concrete is not possible for a given structure, (b) to avoid structure from compressive and tensile forces by volume change due to temperature variations, (c) architectural and functional requirements [1]. Keeping in view the above points, it is impossible to avoid constructions joint in high rise buildings. According to section 3.2.2.2 (ACI 224.3R-95) that the construction joint in columns should be placed at bottom of beams and floor slabs. Similarly for next floor the construction joint should be placed at the top of floor slab [1].
Since beam-column connection is very critical region in reinforced concrete structure. When structure subjected to seismic loading, bending of beams occurs, as a results compressive and tensile forces produce in beam longitudinal bars. These force transfer to joint regions through bond between steel and concrete, produce joint shear demand. With perfect bond and proper development length requirements, it is expected that the plastic hinge may occurs within ½ of the effective depth of beam [2]. According to section 4.5.2.4 ACI 352R-02 the development length of hooked bar should be provided using Equation 1.

$$l_{dh} = \frac{\alpha f_y d_b \ (psi)}{75\sqrt{f_c'} \ (psi)} \qquad (1)$$

When there is construction joint in column, from literature the ductility of connections reduced by 30 to 44%, initial stiffness by 16 to 19% and energy dissipation by 53 to 64% [3]. Similarly in case of reinforced beam having construction joint the load carrying capacity reduces by 15 to 20% as compare to monolithic beam [4]. But in construction of multi storey of Due to very limited research on construction joint at columns in flanged-beam-column connections, this paper covers the corner beam-column connection to supplement the existing information.

II. METHODOLOGY

2.1. Detailing of specimens:

A total of two half scaled specimens were design according to ACI 352R-02 and ACI 318-14. The reinforcement details of both specimen (prototype) is depicted in Figure 1. Both specimens were scaled down using static similitude requirements. Half scale was selected due to availability of resources in structural laboratory.

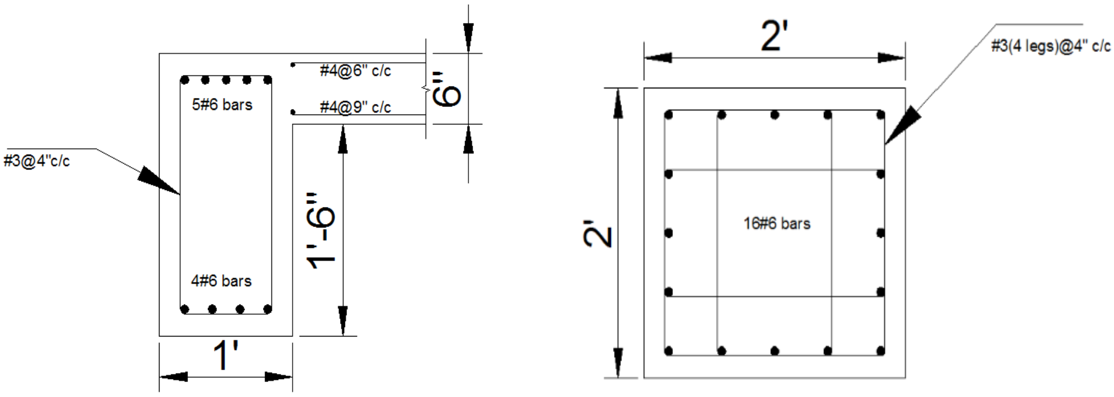

Figure 1: Reinforcement details of both specimens

2.2. Casting of specimens:

Specimen 1 was poured monolithically. Similarly, specimen 2 having construction was casted sequentially as per 3.2.2.2, ACI 224.3R-95. Column was place horizontal and flanged beam was erect vertical to pour concrete. At first, both bottom and top columns were poured and finally joint region along with flanged beam were poured a week later, as depicted in Figure 2.

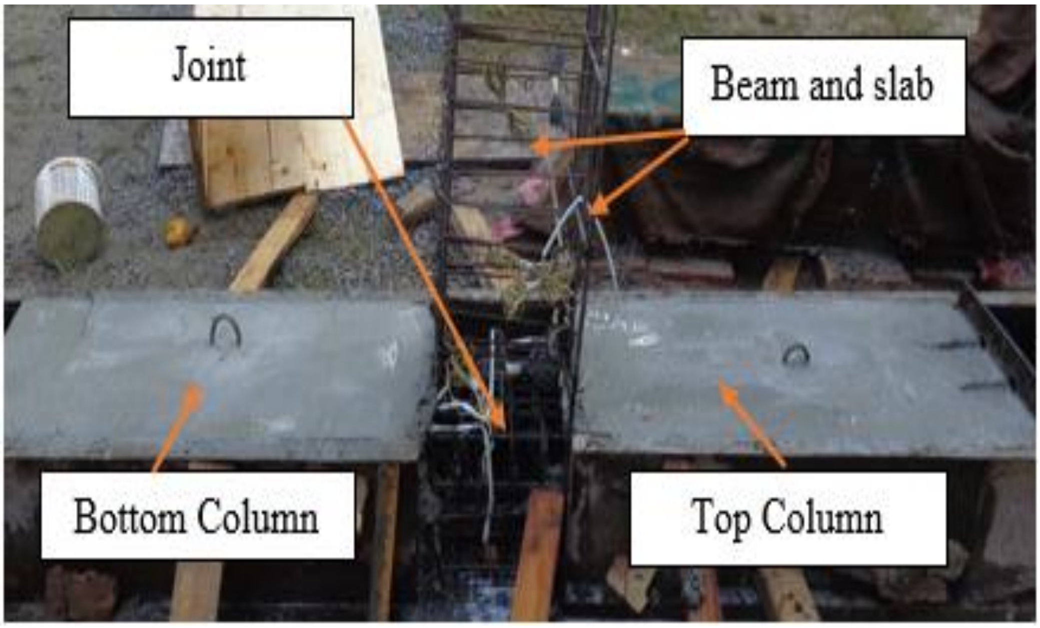

Figure 2: Sequential casting of specimen 2

2.3. Testing setup:

The corner flange-Beam-Column connection was extracted from multi story frame on assumption that point of contra flexure occurs in columns at mid height and in beam at mid span. Based on this assumptions the bottom column was connected with direct pin connection. Similarly, the top column was connected with horizontal roller support in order to allow vertical movement, Figure 3. Similarly column was loaded with constant axial load of Agfc'/6 and full cyclic load was applied at beam free end. The displacement controlled history as shown in Figure 4 was applied at beam tip.

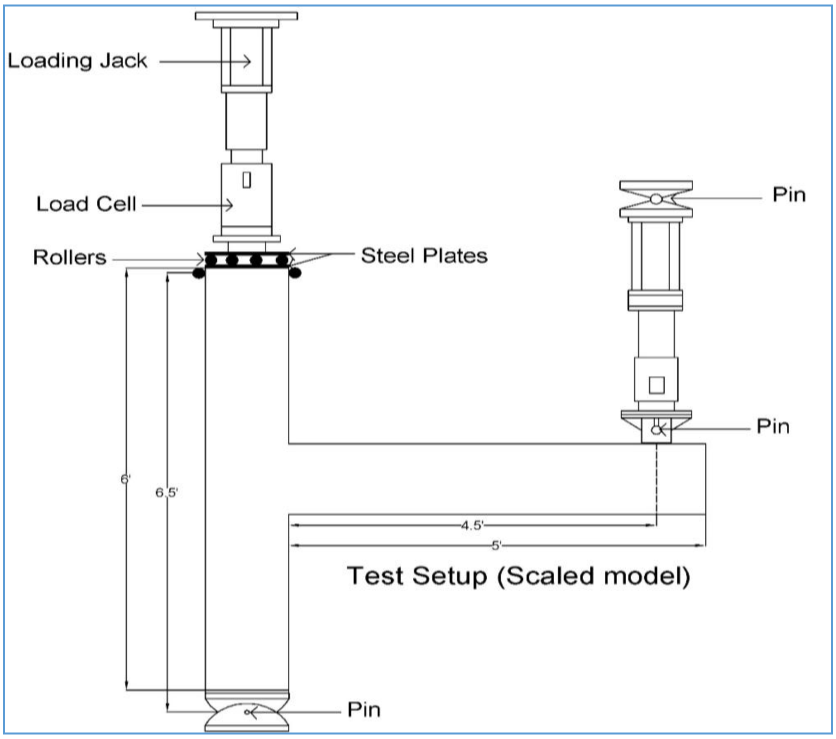

Figure 3: Testing setup

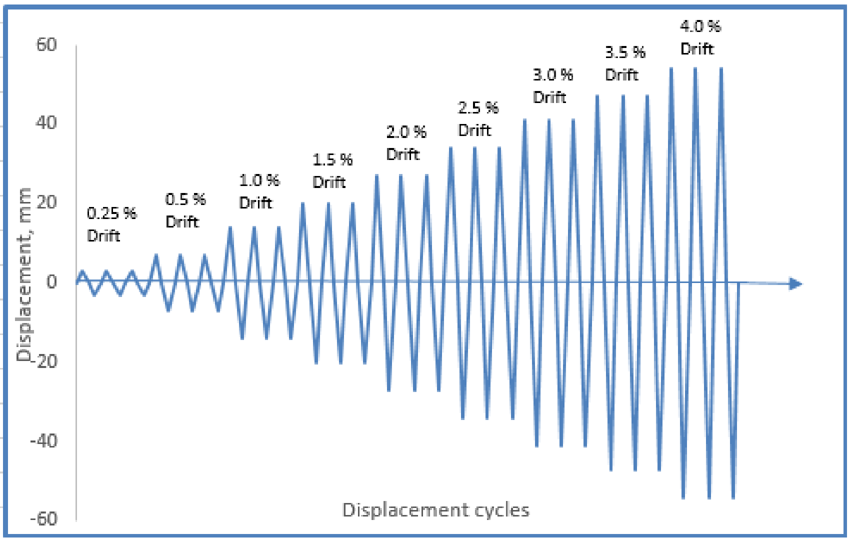

Figure 4: Displacement history for both specimens

III. RESULTS AND DISCUSSION

3.1. Force deformation behavior:

The force-deformation behavior of both specimens is shown in Figure 5. It can be seen from Figure 5 that maximum load carrying capacity of specimen 1 was 33.5 kN and specimen 2 was 24 kN. The first crack starts at 1% drift in specimen 1 while in specimen 2 at 0.5% drift. The maximum load carrying capacity of specimen 1 occurs at 1.5% drift while in specimen 2 at 1% drift. The strength degradation is shown pictorially in Figure 5. It can be seen that due to poor bond strength of concrete with steel reinforcement in the region. Specimen 2 shows no strength degradation after 1.5% drift and continues almost line with negligible variation in strength. This is primarily due to just opening and closing of crack at beam column interface. Similarly specimen 1 maintain their strength till the end of test, shows excellent energy dissipation. The percent difference in strength degradation is shown pictorially in Figure 6. It can be seen from Figure 6 that due to construction joint in specimen 2 the % difference in strength degradation was 54%.

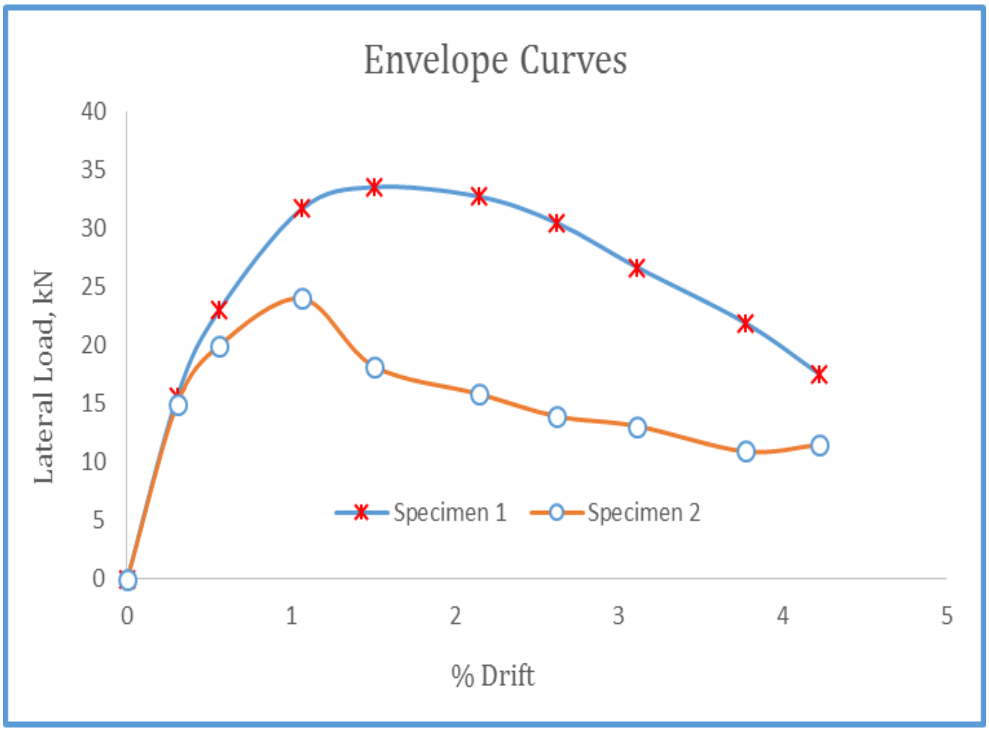

Figure 5: Envelop Curves of both specimens

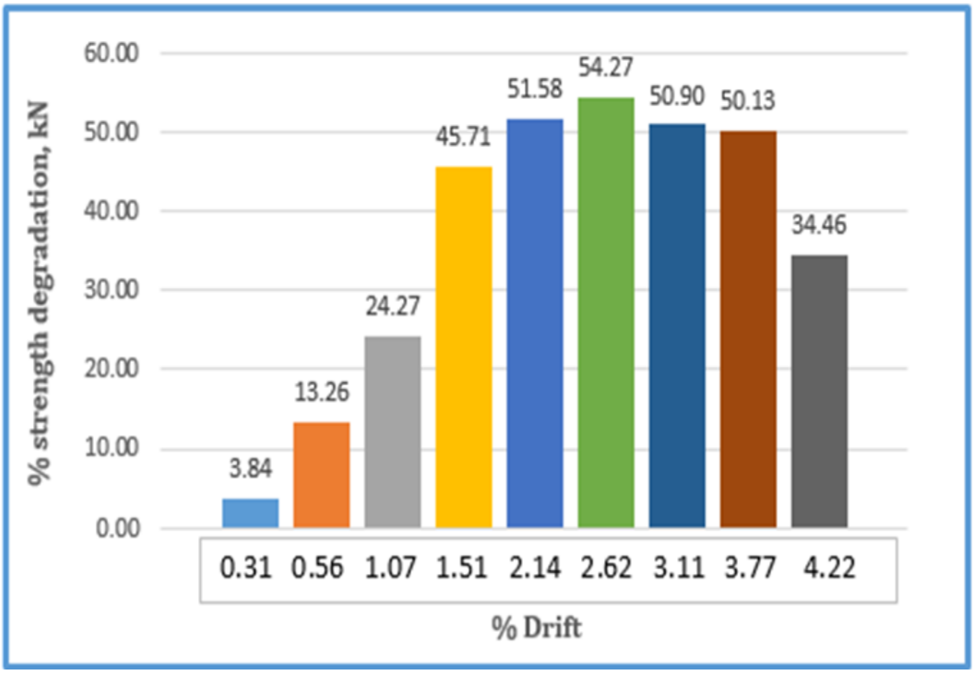

Figure 6: Percent strength degradation with drift

3.2. Stiffness degradation

Stiffness degradation versus % drift relation is shown in Figure 7. It can be seen from Figure 7 specimen 2 shows maximum stiffness degradation as compare to specimen 1. The overall percent stiffness degradation is shown in Table 1. It can be seen from Table 1 that % difference in stiffness degradation due to construction joint was 50% at 3% drift.

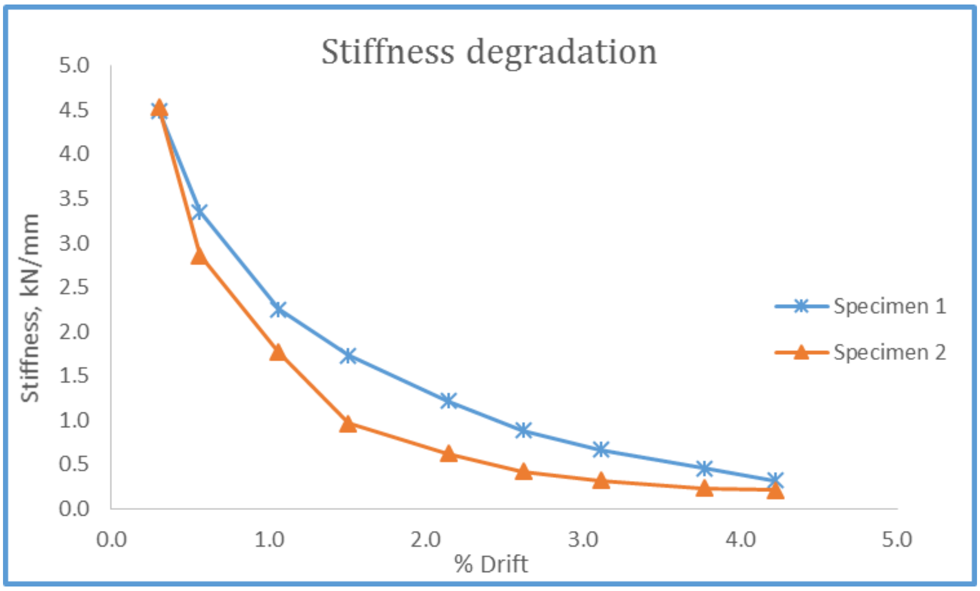

Figure 7: Stiffness degradation of both specimens

Specimen 1 Stiffness kN/mm	Specimen 2 Stiffness kN/mm	% Difference
4.50	4.54	-0.96
3.35	2.86	14.76
2.25	1.76	21.75
1.73	0.96	44.13
1.21	0.62	48.64
0.88	0.42	52.24
0.66	0.32	52.37
0.46	0.23	49.66
0.32	0.21	34.17

Table 1: percent difference in stiffness of both specimens

IV. CONCLUSIONS

- Due to construction joint the connection shows rapid strength degradation after first crack. Similarly, specimen having no construction joint shows very gradual strength degradation.
- The maximum load carrying capacity decreases up to 39% due to construction joint.
- Stiffness degradation in specimen having construction joint increases up to 50%.
- Due to construction joint the connection lost its strength up to 20% (Collapse prevention Level) at 1.5% drift, similarly specimen having no construction joint, 20% strength reduction occurred at 3% drift. Which was primarily due to weak bond between steel and concrete in the joint core.

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