Evaluation of Non-Ductile Reinforced Concrete Building Corner Joint Experiencing Early Column Failure

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ABSTRACT

Column-beam joints of reinforced concrete (RC) buildings designed prior to the adoption of the modern designed code are susceptible to failure when subjected to seismic action. These joints jeopardize the stability of buildings designed before 1970. Failure mechanism of these structures is known, but their vulnerability is unknown. These buildings were designed mainly for gravity loads and they lack adequate lateral load resistance. Their corner joints contain deficient seismic detailing causing a non-ductile behavior. When experiencing lateral forces these buildings behave inelastically causing failure of the joints and ultimately total structural collapse. This research will investigate and evaluate the behavior of one 3D ½ scale corner non-ductile RC beam-column corner joint under constant column compression force and a quasi-static reverse cyclic transverse loading on the beam.

ACKNOWLEDGEMENTS

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1. INTRODUCTION

Column-beam joints of reinforced concrete (RC) buildings designed prior to the adoption of the modern design code are susceptible to failure when subjected to moderate or severe seismic action. These joints jeopardize the stability of buildings designed before 1970. Failure mechanism of these structures is known, but their vulnerability is unknown. These buildings were designed mainly for gravity loads and they lack adequate lateral load resistance. Their corner joints contain deficient seismic detailing causing a non-ductile behavior. Therefore, when experiencing excessive lateral deformation these buildings behave inelastically causing failure of the joints and ultimately total structural collapse.

The deficiencies in seismic detailing were determined by Beres et al. in 1996 and are the following:

- Longitudinal reinforcement ratio in the columns is less than 2%.
- Lapped splices of column reinforcement are furnished just above the construction joint at the floor level where maximum moments and plastic hinges occur.
- Widely-spaced column ties provide little or no confinement to the joint region.
- Little or no transverse reinforcement within the beam-column joint.
- Discontinuous positive beam reinforcement with a short embedment length into the column.
- Construction joints below and above the beam-column joint.
- Columns with bending moment capacity less than that of the beams (weak column strong beam case).

The details in the tested specimen are deficient in the following areas:

- The column's flexural strength is less than the beam's.
- The hooks in the column are bent at 90 degrees.
- There is a lack of transverse joint reinforcement.

Only these three deficiencies were applied to the specimen to prevent longitudinal reinforcement slip failure before assessing the actual joint strength.

The current design code requires a mode of failure in which the plastic hinging occurs in the beam-joint. The hinging would absorb most of the energy stored in the RC structure when subjected to earthquake activity. However, for buildings with deficient detailing the plastic hinge forms in the column-joint creating a brittle mode of failure in the joint before the beam yields. This behavior is undesirable because the joint yields before dissipating much of the seismic energy. Once the joint fails the overall response and stability of the frame is damaged facilitating total structural collapse. Therefore, it is necessary to evaluate the performance of deficient beam-column joints under seismic loading to eventually devise adequate and effective retrofitting techniques that would improve seismic behavior of older RC lateral load resisting moment frames.

1.1 RESEARCH OBJECTIVE AND SCOPE

This research will investigate and evaluate the behavior of one 3D ½ scale corner non-ductile RC beam-column corner joint specimen under a constant column compression force, and a quasi-static reverse cyclic transverse loading on the beam following a pre-defined displacement history. This research will also evaluate the corner joint shear capacity and compare it to the exterior joint shear capacity.

Similar investigations have been conducted to study the importance of beam-column joint on the seismic behavior of deficient RC structures (Beres et al. 1996, El-Amoury and Ghobarah 2002), in some cases they involve 2D beam-column joint subassemblies. My 3D specimen includes a portion of the transverse beam and the slab. The transverse beam adds confinement to the joint increasing the joint shear capacity while the slab adds moment capacity to the longitudinal beam. By adding the transverse beam and the floor slab more realistic results are expected.

2. LITERATURE REVIEW

2.1 INTRODUCTION

The importance of understanding the behavior of non-ductile RC structures beam-column connections has been especially crucial with the damage caused by recent earthquakes. When these types of buildings are subjected to seismic action it is observed that the most critical element of the structure is the beam-column joint. In recent decades a number of experimental and analytical studies have been done to better understand the behavior of beam–column joints. Most of the studies investigate the shear behavior of the joints, but additional data is necessary to accurately assess the behavior of lightly transverse reinforced joints subject to early column failure.

2.2 BEAM COLUMN CONNECTIONS

The following is the ACI definition for a beam-column connection in a monolithic reinforced concrete structure:

> A beam-column joint is defined as that portion of the column within the depth of the deepest beam that frames into the column…. A connection is the joint plus the columns, beams, and slab adjacent to the joint. A transverse beam is one that frames into the joint in a direction perpendicular to that for which the joint shear is being considered (ACI 352).

As previously mentioned current design code enforces all plastic hinges to form in the beam in order for the structure to absorb most of the seismic energy through inelastic deformation. Current design suggests that column hinges should be avoided because they result in a high ductility demand and can cause collapse of buildings. Many investigators have studied the effects of varying relative beam to column flexural strength ratios, M_r. For example, Ehsani and Wight 1985a, 1985b, Durrani et al. 1987, French and Moehle 1991, and Di Franco et al. 1995. After evaluating the results from these experiments, ACI 352 R-02 (2002) announced that the ratio of the sum of the flexural strengths of the column sections connecting to the joint divided by the sum of the flexural strengths of the beam sections connecting to the joint should not be less than 1.2. This prevention assures that plastic hinges occurs in the beam creating a “strong column weak beam” structural system. The relative beam to column flexural strength ratios for buildings built before 1970 are not greater than 1.2. For the specimen tested on this research this ratio was approximately 1 creating a “weak column strong beam” structural subassemblies. The specimen detailing and design disqualify it as a seismic type 2 connection.

After testing two beam-column joints of low column axial load, in 1974 Megget concluded that the reinforcing in the transverse beams adds little confinement to the connection region. Though, the reinforcement didn’t add much confinement the actual transverse beam had a great contribution to the joint confinement. This helped to strengthen the joint moving the plastic hinge away from the joint into the beam.

The ACI 352-02(2002) suggests that when evaluating the beam’s flexural strength the slab should also be considered. When a building is subjected to earthquake motion a portion of the slab flexural reinforcement interact with the beam’s reinforcement to take the load. Therefore, to acquire more realistic results from the research the slab should be included in the specimen.

Reinforce concrete structures designed before 1970 are primarily intended to support gravity loads and they lack capacity for lateral loads. Due to a deficiency in the reinforcing detailing in the joint region and other members, these structures are commonly characterized as non-ductile. In 1996, Beres et al. conducted a study to evaluate the performance of these beam-column joints when subjected to seismic action.

For a more complete literature review regarding beam-column joints and a graphical comparison in the strength of joints subjected to different axial loads consult Wael Hassan's dissertation paper.

2.3 QUASI-STATIC CYCLIC LOADING

When a specimen is subjected to a quasi-static cyclic loading it undergoes a numbers of displacement-controlled cycles as shown in Figure 1.1. These loading cycles are applied slowly in order to eliminate the effects of material strain rate (Sin 2004) creating conservative estimates of the real strength of the structure or structural assembly. In contrast, when a specimen is subjected to a dynamic load it experiences an increase in the strain rate resulting in an overall strength increase. Researchers have applied quasi-static loading cycles in terms of displacement ductility or interstorey drifts. Displacement ductility is defined as the ratio of the maximum displacement to the displacement reached at yield. Interstorey drift is defined as the interstorey horizontal displacement divided by the storey height. Interstorey drift is commonly used since it avoids the difficulty of defining the yield displacement level. When applying interstorey drift as a test criterion it is important to understand that the imposed interstorey drift always depends on the stiffness of the structure and the level of ductility factor to be imposed (Park 1989).

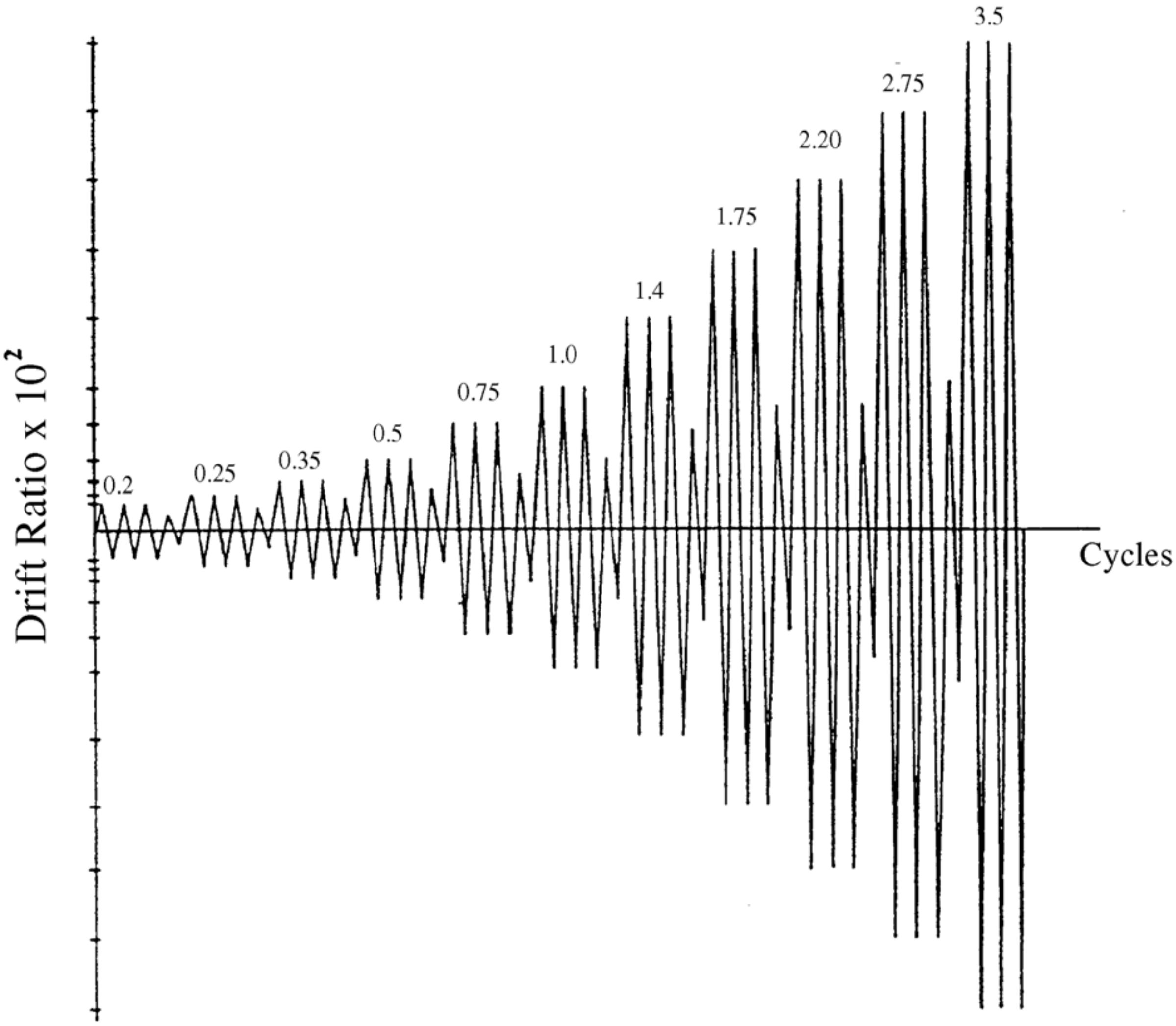

Figure 1.1: Example of test sequence of displacement controlled cycles (ACI T1.101

ACI T1.1 01 (2001) denotes the guidelines for testing of structural moment frames. It recommends an interstorey drift as that shown in Figure 1 for a test sequence. This sequence ensures that displacements are increased gradually in intervals that are neither too large nor too small. Too large displacement intervals can make it very difficult to read with accuracy the drift capacity of the system. In contrast, if the steps are too small the system may be softened by loading repetitions and also the results can cause undesirable brittle failure modes.

3. EXPERIMENTAL STUDY

3.1 DESCRIPTION OF TEST UNIT

In a joint effort to better understand the behavior of older building's beam-column connections, PEER collaborated with the National Center for Research on Earthquake Engineering (NCREE) to design a RC 2D frame to be tested dynamically in Taiwan. The specimen investigated in this research was a replica of a corner section of the NCREE frame as displayed below in Figure 3.1. The specimen was reproduced assuming that the points of contraflexure occurred in the mid-height of the columns and mid-span of the beam.

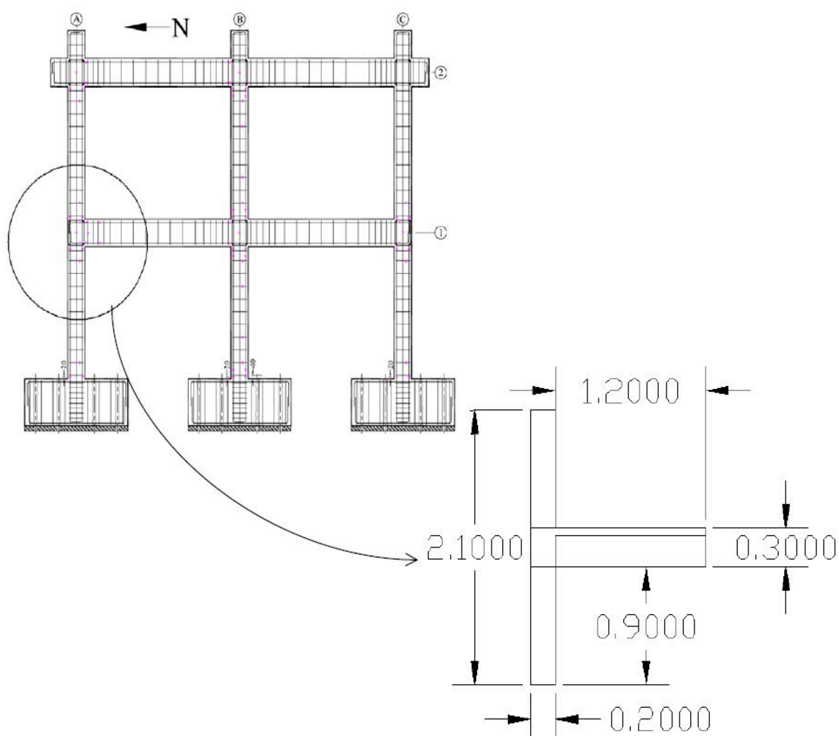

Figure 3.1: 2D Frame to be tested by NCREE with a close up of the location and dimensions of the specimen

The unit was a half-scale model of a typical corner beam-column joint found in RC buildings design before 1970.

The corner RC beam-column subassembly includes the floor slab, transverse stub, longitudinal strong beam and weak column as shown in Figure 3.2. There is no transverse joint reinforcement and the hooks of the column hoops are bent at 90 degrees not providing any joint confinement.

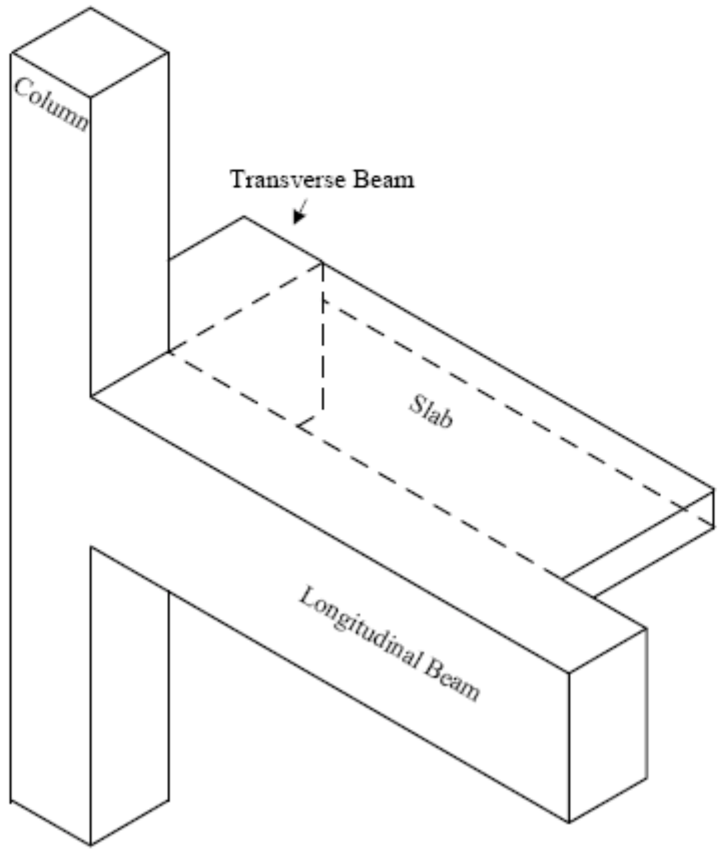

Figure 3.2: 3D View of Specimen

3.2 MATERIAL PROPERTIES

3.2.1 Concrete

The design required a concrete compressive strength of 4000 psi and a maximum aggregate size of 3/8”. The concrete design and mixture was done by Lev, the concrete technician in Davis Hall at the University of California at Berkeley. While casting the subassembly, 9 6x12 concrete cylinders samples were made. The concrete cylinders were removed from the plastic molds one week later at the same time that the form work was removed from the specimen. All 9 cylinders were cured in the same manner as the specimen. Three cylinders were tested for maximum compressive strength at 7, 14 and 22 days after casting. From the cylinder tests the actual concrete strength during testing was closer to 5600 psi. The pictures of the casted test cylinders and compression test machine are displayed in Figure 3.3 and Figure 3.4 respectively. A summary of the tests results are given in Table 3.1.

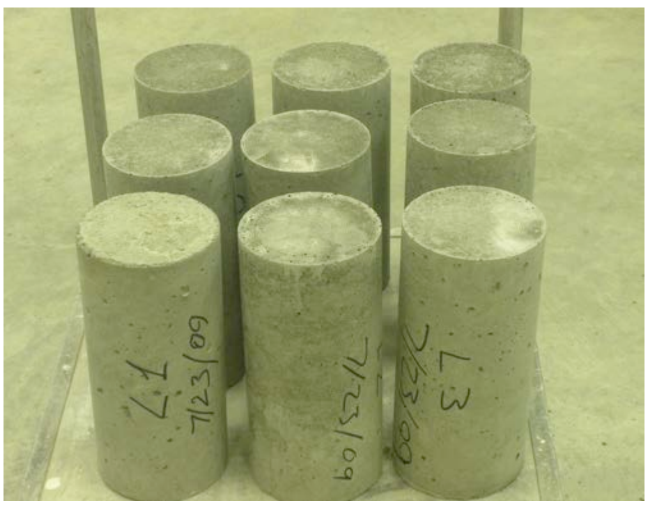

Figure 3.3: Test cylinders used for concrete compressive strength testing

Figure 3.4: Compression machine used to test concrete cylinders

	number	strength (lbf)	stress (psi)
7 days	1	103000	3643.44
	2	96900	3427.66
	3	97500	3448.89
Average		99133	3506.66
14 days	4	120700	4269.54
	5	130200	4605.59
	6	130000	4598.51
Average		126967	4491.22
22 days	7	158900	5620.80
	8	157200	5560.67
	9	160900	5691.55
Average		159000	5624.34

Table 3.1: Compressive strength of concrete

3.2.2 Steel Reinforcement

The reinforcement in the specimen consisted of five different sizes of rebar. #2 smooth bars were used for the slab flexural reinforcement. The original design details called for W7 with a diameter of .298in for the column hoops, but due to a scarcity of such wire it was replaced by a w6.5 with a diameter of .288in. The spacing of the hoops was recalculated to maintain the original confinement pressure and shear capacity. The spacing was altered from 1.57in to 1.47in center to center. #3 bars were used for the beam stirrups and slab transverse reinforcement. The column was reinforced with #4 bars in the longitudinal direction. The longitudinal beam and the transverse stub flexural reinforcement consisted of #5 bars.

The mechanical properties of the steel bars were acquired by conducting tensile tests on 3 bars of each size. 24in long samples were used for most of the bars; except for #4 16in long samples were used due to scarcity of the same material. A summary of the mechanical properties of the reinforcement used are display below in Table 3.2. The tensile test machine used is displayed in Figure 2.4.

Rebart Size/Diam.(in)	Yield Strength (fy) (ksi)	fy avg (ksi)	Yield Strain (єy) (%)	Єyavg (%)	Ultimate Strength (fu) (ksi)	fu avg (ksi)	Ultimate Strain (єu) (%)	єuavg (%)
#2/.25	56.096 52.390 50.523	53.003	0.259 0.181 0.200	0.213	71.768 71.061 70.580	71.136	22.707 20.151 12.899	18.586
w6.5/.288	50.534 56.895 57.063	54.831	0.276 0.219 0.208	0.234	76.865 73.935 75.810	75.537	1.582 2.639 1.291	1.837
#3/.375	67.044 67.545 67.005	67.198	0.314 0.262 0.292	0.289	100.193 100.424 100.103	100.240	14.937 14.080 12.680	13.899
#4/.5	73.147 70.092 71.287	71.509	0.501 0.665 0.964	0.710	112.886 108.890 110.219	110.665	9.956 9.132 10.750	9.946
#5/.625	66.268 65.707 65.707	65.894	0.282 0.282 0.618	0.394	107.519 105.836 107.355	106.903	10.931 10.391 10.837	10.720

Table 3.2: Actual steel reinforcement strength based on tensile


Figure 3.5: Tensile test machine used to acquire steel

3.3 CONSTRUCTION OF TEST UNIT

3.3.1 Specimen Detailing

The specimen subassemblies featuring a corner connection are shown in Figure 3.6 followed by the cross-sectional detailing of the column, beam and transverse beam in Figures 3.7, 3.8 and 3.9 respectively.

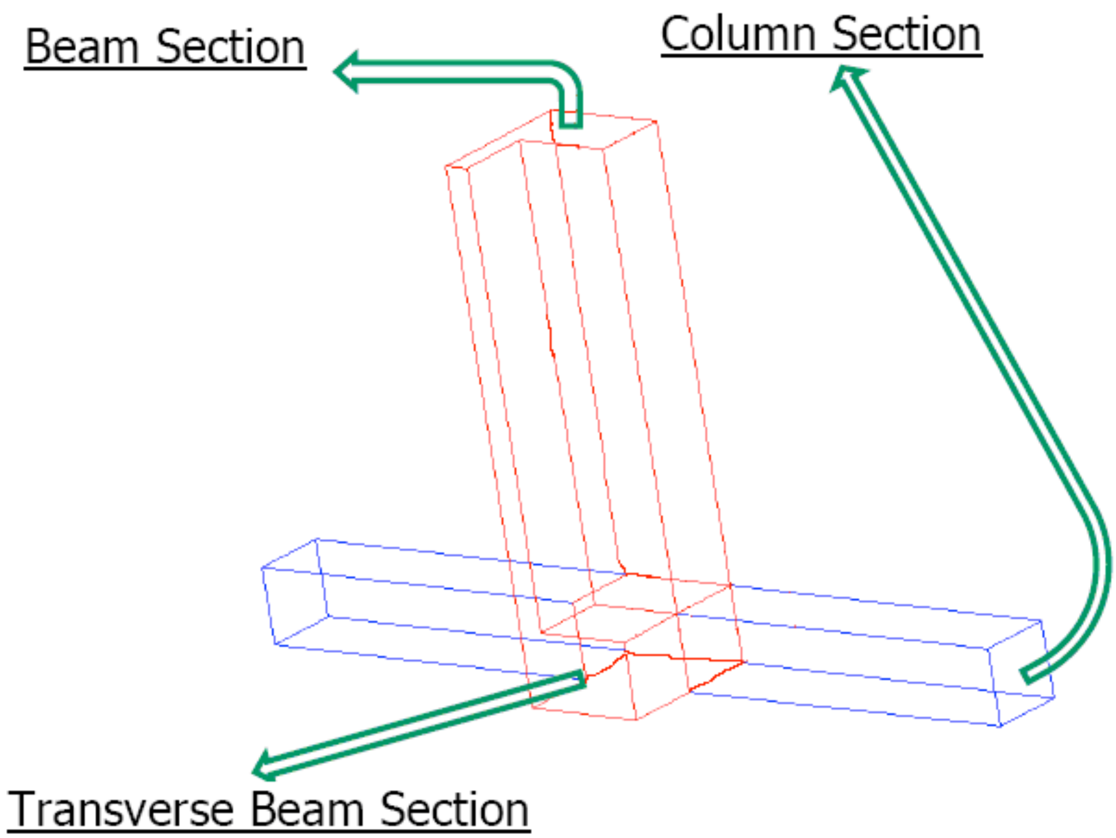

Figure 3.6: Specimen

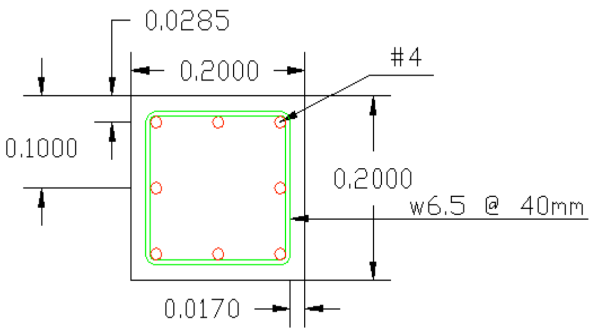

Figure3.7: Column detailing

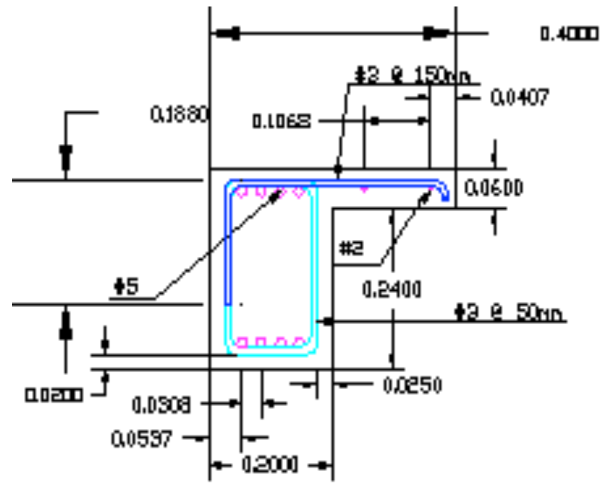

Figure3.8: Beam detailing

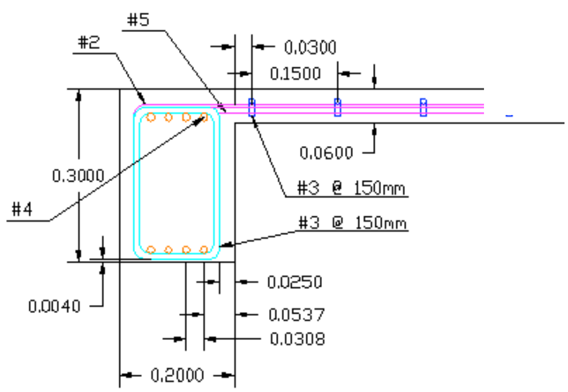

Figure 3.9: Transverse beam detailing

The beam had a total length of 1.2 meters while the column length was 2.1 meters. The column hoops ended with 90° bent hooks for non seismic detailing.

3.3.2 Concrete Form

The concrete form consisted of the column laying on the platform while the beam stood vertically. Two polyvinyl chloride pipes with a diameter of 1¼" were installed through the slab section of the form adjacent to the longitudinal beam to leave two holes which then accommodated the thread rod used for attaching the actuator to the longitudinal beam. Additionally, two 1" holes were made in the center of the joint in the column facing the platform and 2 more in the center of the joint on the free side of the column facing apposite to the transverse beam, to allow for the placement of shear devices.   A picture of the longitudinal beam form is displayed in Figure 3.10. Figure 3.11 shows the concrete form for the column and the transverse beam. The concrete form for the assembly is display below in figure 3.12.

Figure 3.10: Concrete form for the beam

Figure 3.11: Concrete form for the column & transverse beam

Figure 3.12: Concrete form for the beam column specimen

3.3.3 Steel Reinforced Bar Cage

The longitudinal beam together with the slab reinforcing cage was built separately from the column and once they were assembled together the transverse beam's reinforcing was added. Pictures of the longitudinal beam, beam and slab, column and joint including the transverse beam reinforcing are display below in Figures 3.13, 3.14, 3.15 and 3.16 respectively.

Figure 3.13: Longitudinal beam reinforcement

Figure 3.14: Longitudinal beam & slab reinforcement

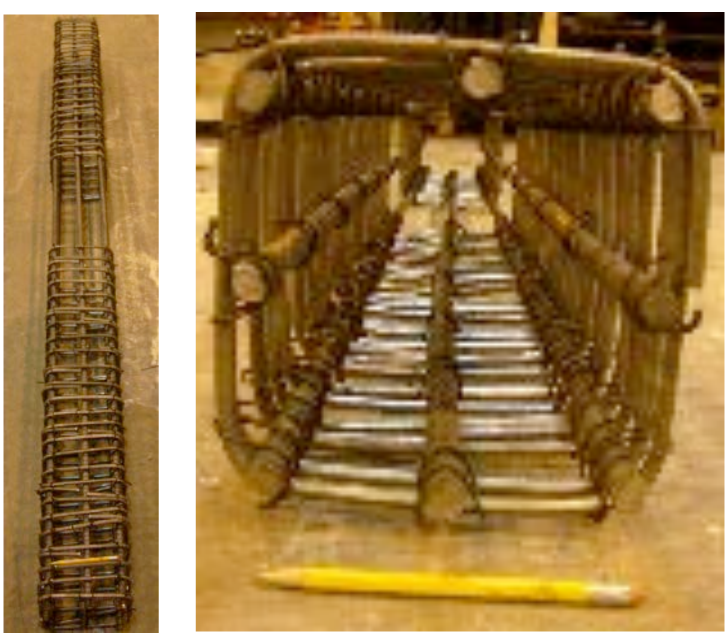

Figure 3.15: Column reinforcement

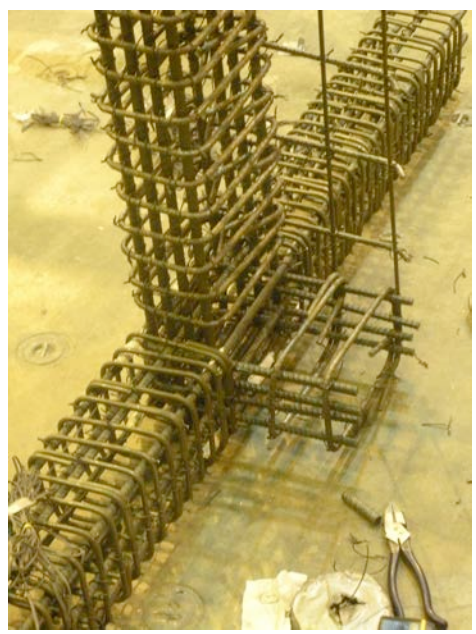

Figure 3.16: Joint reinforcement

The hoops in the column were arranged so that the bent end alternated in all four corners.

However, the bent ends in the beam's stirrups only alternated between the two top (side of the slab) corners. This was done in a consistency manner to better simulate construction practice for older buildings.

3.3.4 Pre-Casting Instrumentation

There were four shear devices fabricated and were used to attach displacement transducers. Two of them were installed vertically beneath the center of joint and the other two were placed horizontally on the side of the joint. The devices were made of modified ½" thick square stock covered in Teflon tape to prevent concrete confinement. The bottom part was surrounded with foam and it was placed into a 1" diameter tube to provide room for the devices to move freely during joint deformation. Figure 3.17, 3.18 and 3.19 show the four shear devices before being installed, the two devices on the bottom of the joint and the side of the joint shear devices respectively.

Figure 3.17: Shear devices

Figure 3.18: Bottom of joint shear devices

Figure 3.19: Side of joint shear devices

Additionally, 10 strain gauges were installed in the column's corner longitudinal reinforcing bars. There were two strain gauges installed in each bar approximately 1" outside the joint. One strain gauge was installed in the middle of the join on two opposing diagonally bars. Figure 3.20 show the position of the strain gauges in the exterior face of the join. The four strain gauges outside the joint where place identically in the opposite side of the joint, while the middle gauge was placed in the bottom bar.

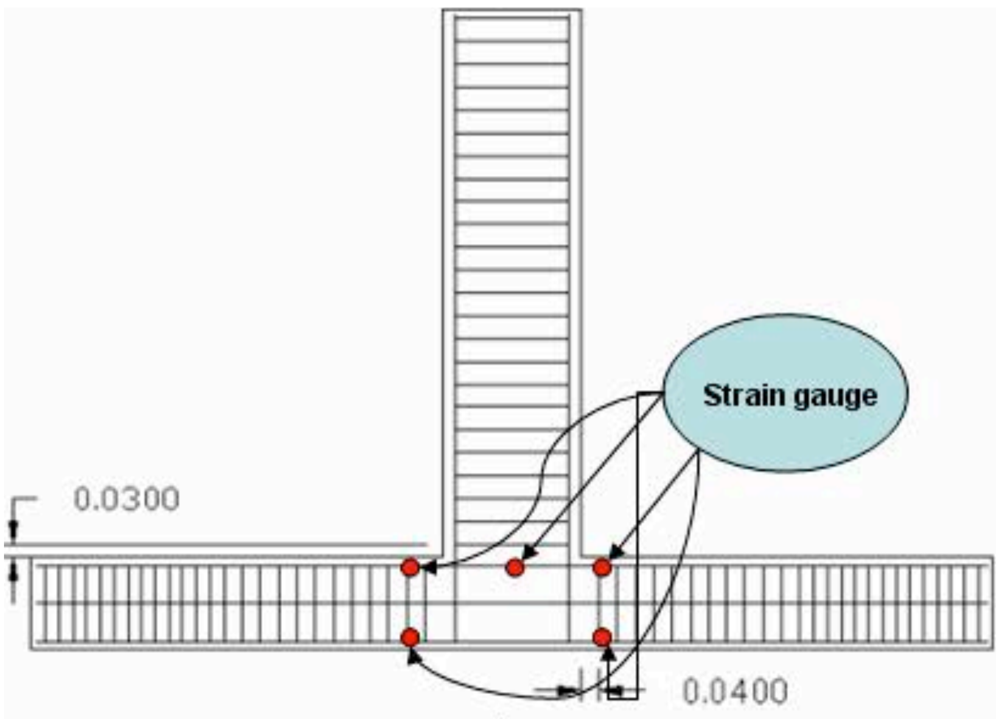

Figure 3.20: Strain gauges location

3.3.5 Concrete Casting and Grid

The concrete design and mixing was done by the concrete technician of the Civil and Environmental department at Berkeley University. A total of .256 cubic meters were needed to cast the specimen and the 9 test cylinder. This quantity was acquired using one concrete batch.

While casting a high-frequency vibrator was used to ensure that the concrete fills the entire form. An anchor was placed on top of the beams to allow for a more adequate transportation of the specimen. Workability of the concrete was measured using the slump test which indicated a value of 8”.

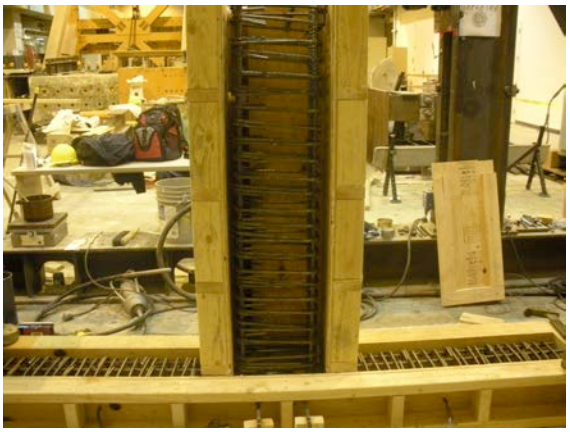

Figure 3.21: Unit before casting

Figure 3.22: Unit and cylinders after casting

Figure 3.23: Specimen after removing the form

After removing the form it was noted that some rock pockets were formed in the slab near the column. These packets were patched using quick drying cement. After patching the specimen, it was painted and a grid was drawn. Spacing between each grid was 50 mm, dividing the column width into four divisions. Figure 3.24 shows the grid lines on the specimen.

Figure 3.24: Grid lines on the specimen

4. TEST SETUP

4.1 INSTRUMENTATION

4.1.1 Displacement Transducers

Ten displacement transducers were installed on the specimen to monitor movement during testing. Two displacement transducers were attached to each side of one end of the column to measure the column in-plane movement and two load cells to measure the column's axial compressive load (see Figure 4.1 and 4.2). Two displacement transducers were attached to the shear devices on the bottom of the joint and two more to the devices on the side of the joint to measure the joint's shear stain (see Figure 4.3 and 4.4). The displacement of the beam in the direction of loading was measured by a displacement transducer attached to the plate on the side of the beam (see Figure 4.5 and 4.6). Another displacement transducer was attached to the loading plate on the side of the column onto the wall perpendicular to the direction of loading to measure out-of-plane motion (see Figure 4.7).

Figure 4.1: Displacement transducer-column in-plane movement

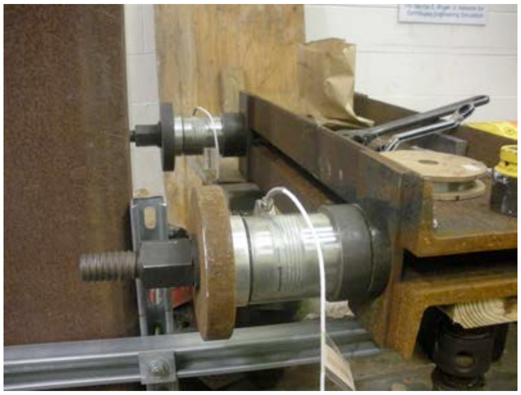

Figure 4.2: Load cells- column axial compressive load

Figure 4.3: Displacement transducers- shear strain on bottom of join

Figure 4.4: Displacement transducers- shear strain on side of join

Figure 4.5: Target for beam in direction of loading displacement transducers

Figure 4.6: Displacement transducers- beam in direction of loading

Figure 4.7: Displacement transducers- beam out-of-plane

4.1.2 Strain gauges

10 strain gauges were installed in the column's corner longitudinal reinforcing bars. There were two strain gauges installed in each bar approximately 1" outside the joint. One strain gauge was installed in the middle of the join on two opposing diagonally bars. The strain gauges in the column outside the joint will aid when trying to determine if the column or the joint fail first. The strain gauges installed in the middle of the joint will assist us in determining if there was yield penetration into the joint. Figure 4.8 show all ten strain gauges connected to the adapters which are connected to the data acquisition computer system.

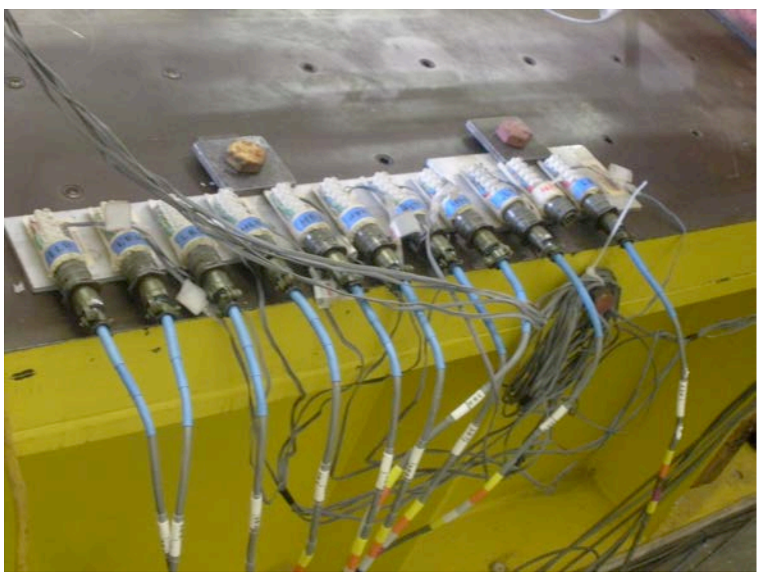

Figure 4.8: Strain gauges connected to data acquisition computer system

4.2 LOADING

4.2.1 Column axial load

The column was supported by two 12” diameter steel rollers on each end to simulate point of contraflexure. Two hollow square sections where attached on top of the column, directly above the rollers, using four thread rods to secure the column on top of the rollers. The column was subjected to a constant axial load of 20% of its axial capacity. This axial compressive force was produced using a manual hydraulic pump and it was transferred to the column using two, parallel to the column, steel rods (Figure 4.9 and 4.10). The rods spanned from a hollow steel section attached to the frame on one side to a two welded together channels on the other side. Two 1” thick steel square plates were placed on each side between the column and the sections to prevent local yielding. The pressure was measured using two load cells, one on each steel rod (Figure 4.11).

Figure 4.9: Compression pump

Figure 4.10: Compression pump gage

Figure 4.11: Compression measuring load cells

4.2.2 Transverse load

The quasi-static reverse cyclic load was applied transversely to the longitudinal beam. The load was applied on the slab side of the beam (see Figure 4.13). To prevent local yielding and to distribute the pressure one 1" thick steel plates were placed on each side of the beam. To allow for rotation, the actuator exerting this force was pinned connected to the frame. The axial compressive force on the beam coming from its on weight and the weight of the actuator was counteracted by using a balanced system. This system consisted of an elevated horizontal, attached to the roof, I-beam to which the top of the specimen's beam was attached using straps onto one side while heavy lead was suspended from the other end of the I-beam (see Figure 4.12). This system was implemented to test under a more realistic scenario.

Figure 4.12: Balance system

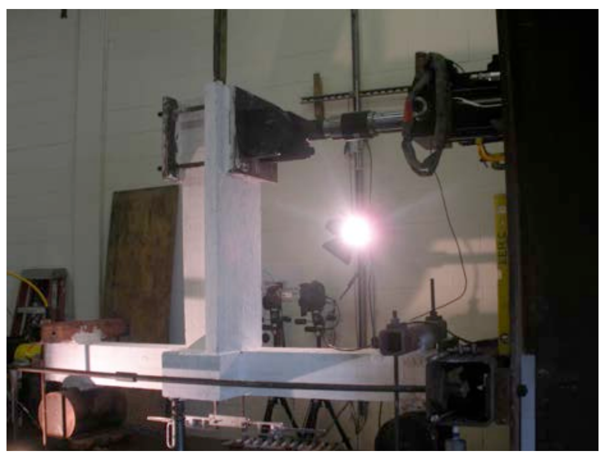

Figure 4.13: Actuator position

4.3 TEST PROCEDURE

The column was constantly subjected to an axial compressive load of *0.2f'c.* The axial compressive load was maintained by manually adjusting the hydraulic pump. The transverse load was applied cyclically with a loading procedure consisting of displacement-controlled steps. The steps were 0.1% drift followed by steps of 0.25%, 0.50%, 0.75%, 1.0%, 1.5%, 2.0%, 3.0%, 5.0%, 7.0%, and 10.0% drift with two cycles per step. The cycles always began with the loads causing tension in the slab. Each drift step consisted of 2 cycles of push and pull. Table 4.1 displays the specification of the actuator while applying the load for all runs of the test.

drift	displacement	Velocity	Duration 1 cycle	Frequency
(%)	(in)	in/sec	sec	Hz
0.25	0.118	0.02	23.625	0.0423
0.5	0.236	0.02	47.25	0.0212
0.75	0.354	0.02	70.875	0.0141
1	0.473	0.02	94.5	0.0106
1.5	0.709	0.02	141.75	0.0071
2	0.945	0.02	189	0.0053
3	1.418	0.02	283.5	0.0035
5	2.363	0.02	472.5	0.0021
7	3.308	0.02	661.5	0.0015
10	4.725	0.02	945	0.0011

Table 4.1: Actuator specifications for each drift

5. EXPERIMENTAL RESULTS AND DISCUSSION

5.1 THEORETICAL PREDICTIONS

The shear capacity was calculated before testing of the specimen in order to anticipate joint shear failure. The shear yielding capacities for the beam and joint due to the transverse force on the beam were calculated using XTRACT a cross-section analysis computer software. From the XTRACT results the beam's yielding moment capacity is approximately 730 kip-in as shown in Figure 5.1. This means that the yielding transverse load is approximately 15.45 kips.

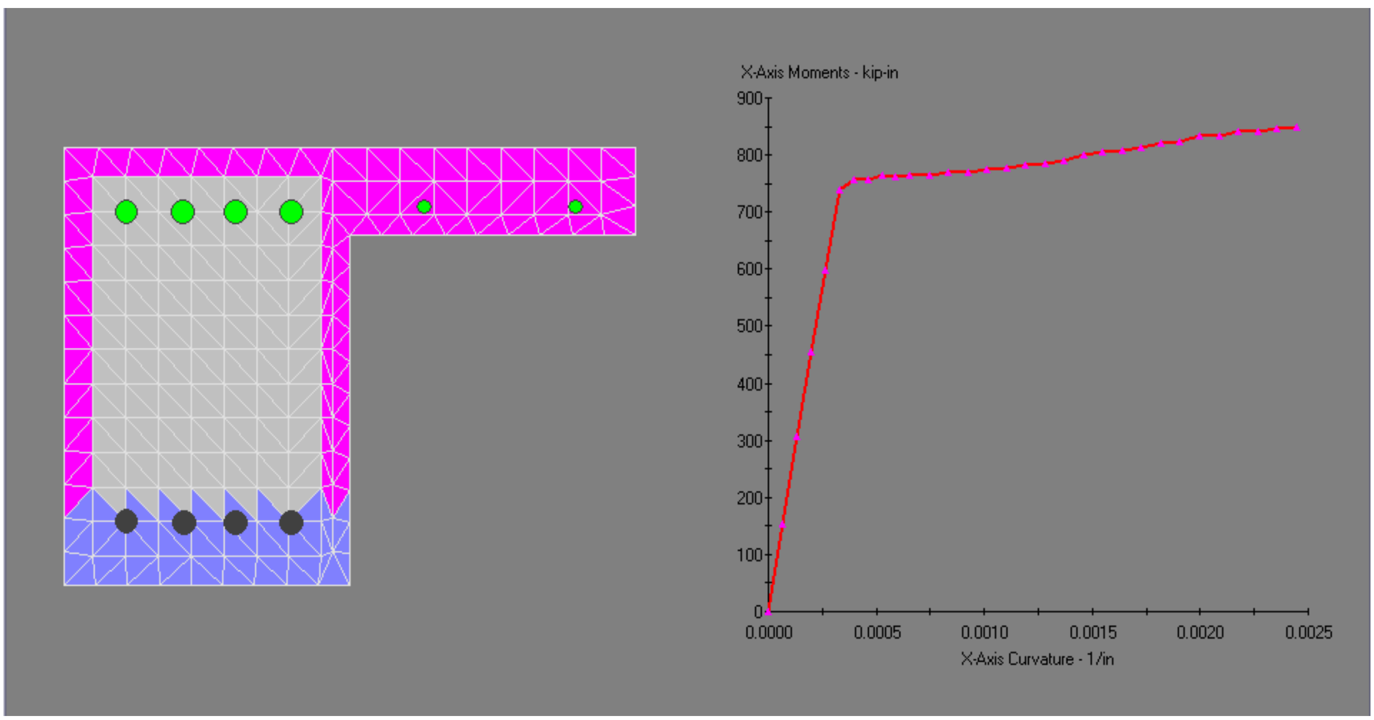

Figure 5.1: XTRACT output of beam's moment capacity

Figure 5.2 illustrates the columns interaction diagram. It shows that at a constant compressive force of 50 kips, the maximum moment capacity is approximately 370 kip-in. The column's moment capacity XTRACT output in Figure 2.3 shows that the yielding moment in the column is approximately 370 kip-in, agreeing with the interaction diagram. This means that the maximum yielding transverse load in the column is approximately 10.45 kips.

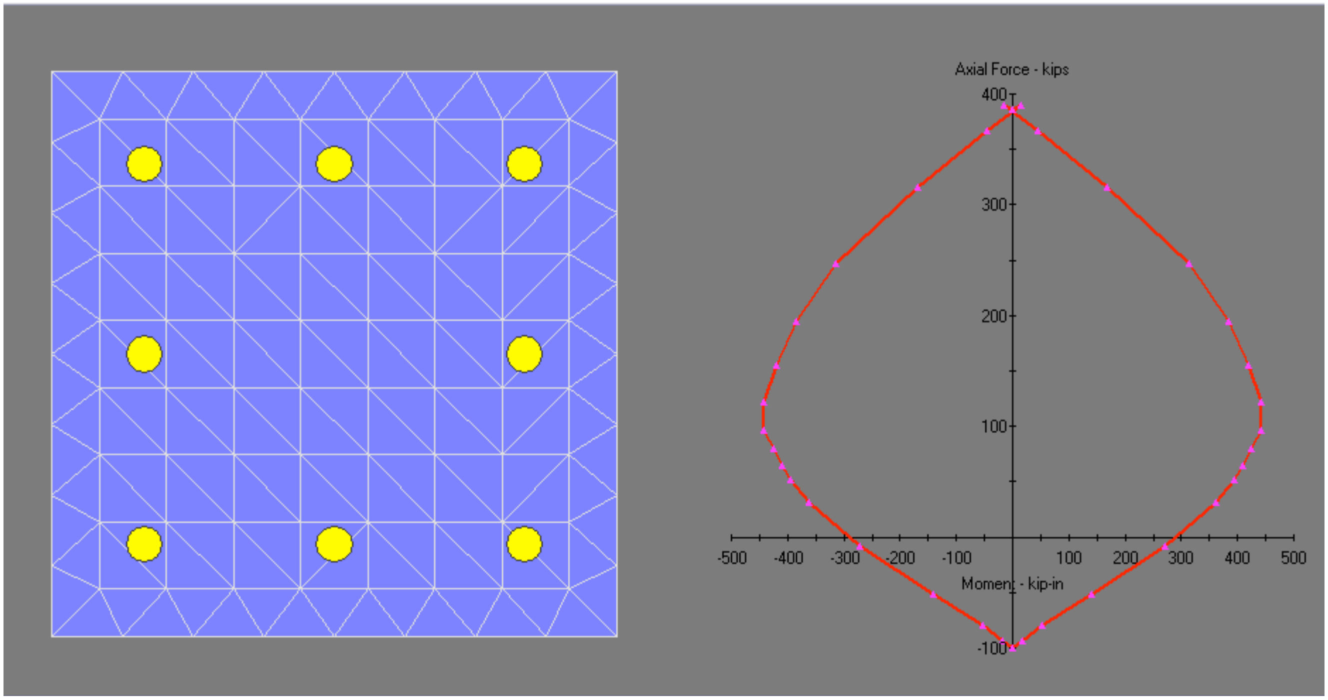

Figure 5.2: XTRACT output of column's interaction diagram

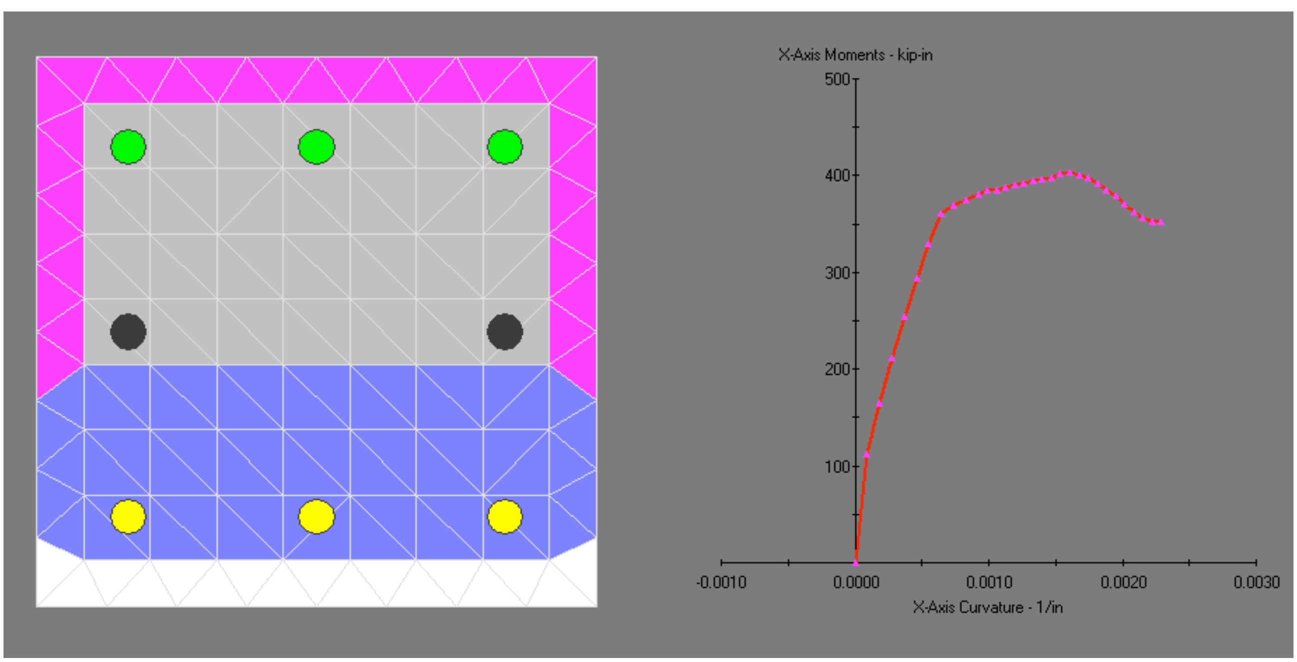

Figure 5.3: XTRACT output of column's moment capacity

5.2 EXPERIMENTAL RESULTS

The transverse applied beam load versus beam lateral displacement is shown in Figure 5.4. The horizontal axis shows the displacement while the vertical axis shows the applied load. It should be noted that a positive displacement indicates tension in the slab side of the beam while a negative displacement indicates compression in the slab side of the beam. The maximum

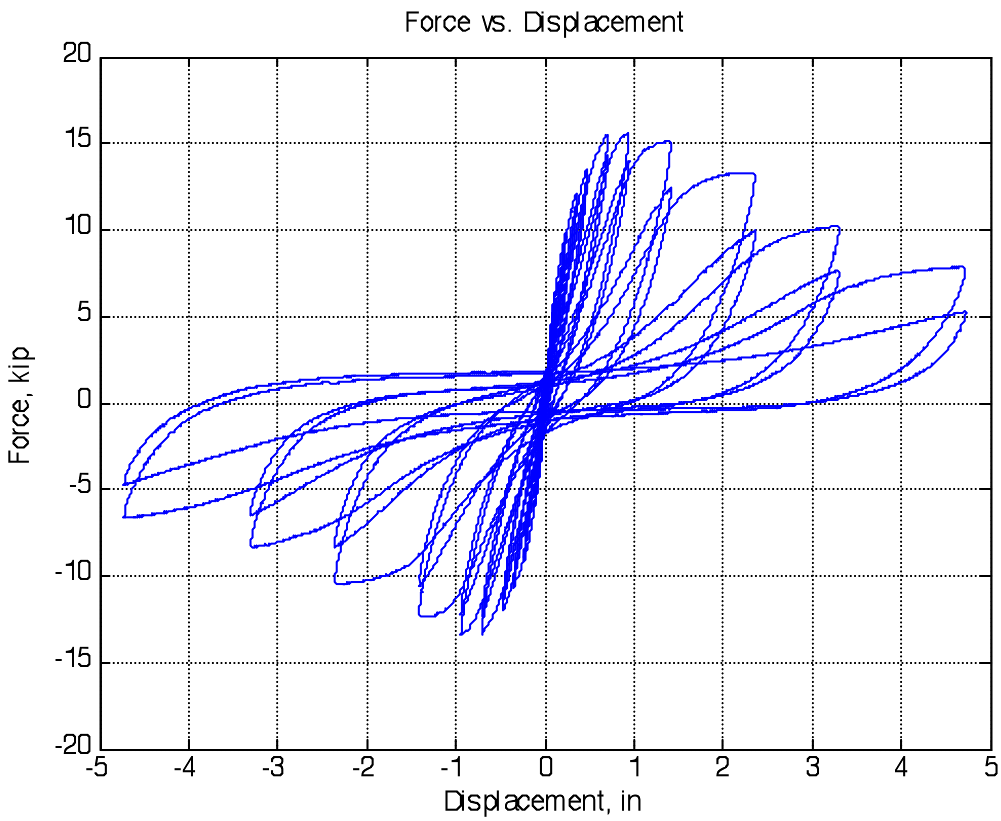

Figure 5.4: Applied force vs. Displacement

positive force applied on the beam was 15.59 kips and it occurred during the 2% drift. While 13.44 kips was the maximum applied load in the negative direction and it also occurred during the 2% drift. Figure 5.5 shows the applied force versus drift for all runs. Thus, the ratio between lateral load capacity in the strong and weak directions is equal to

1.16. The longitudinal reinforcement in the column first yielded during the 1.5% drift at 15 kips.

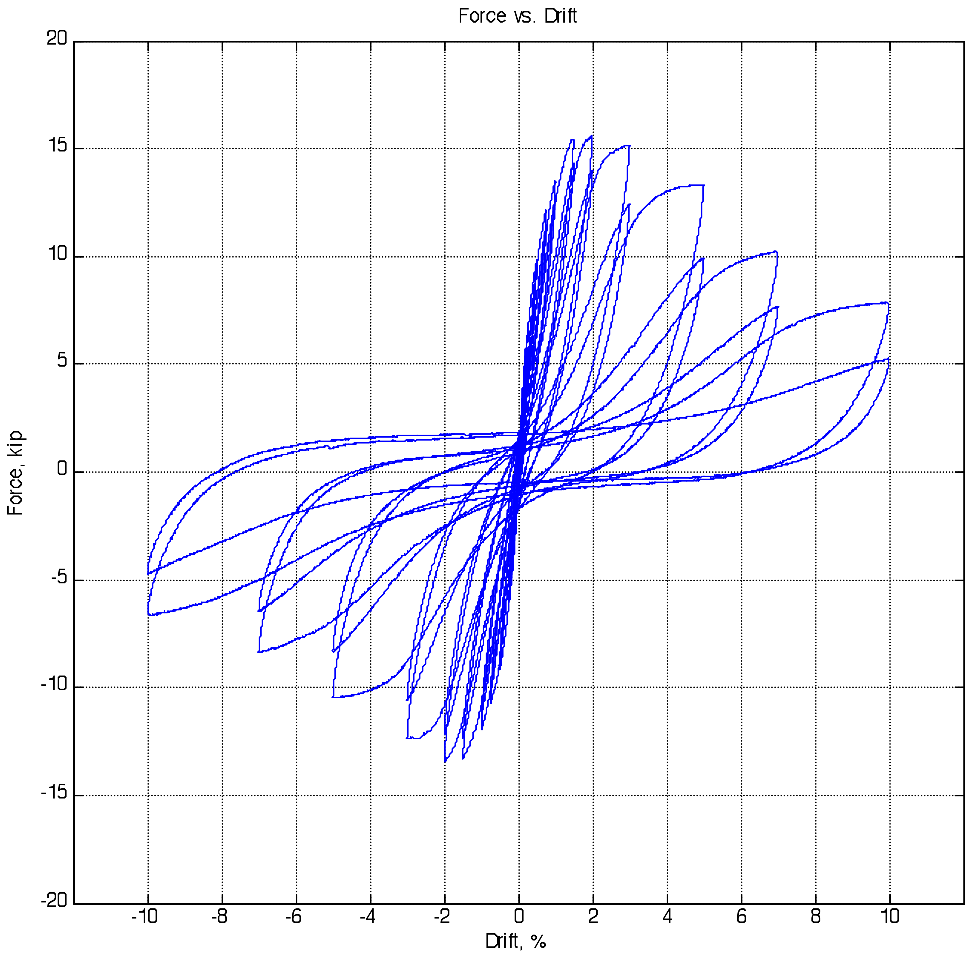

Figure 5.5: Applied force vs. Drift

Figure 5.6 shows damage progression of the joint from the first signs of cracking to significant spalling and ending with joint failure.

Figure 5.6: Damage progression leading to collapse

5.3 DISCUSSION

Figure 5.7 shows the lateral load applied versus drift for an exterior joint with similar detailing and subjected to the same constant compression force as the one studied in this research. The seismic performance of the exterior joint was done by M. Barnes et al. From this graph it is noted that the maximum load in the positive direction occurred at a drift ratio of 1.5% with a value of 17.5 kips; in the negative direction, the maximum load occurred at a drift of 2% with a value of 16.5 kips.

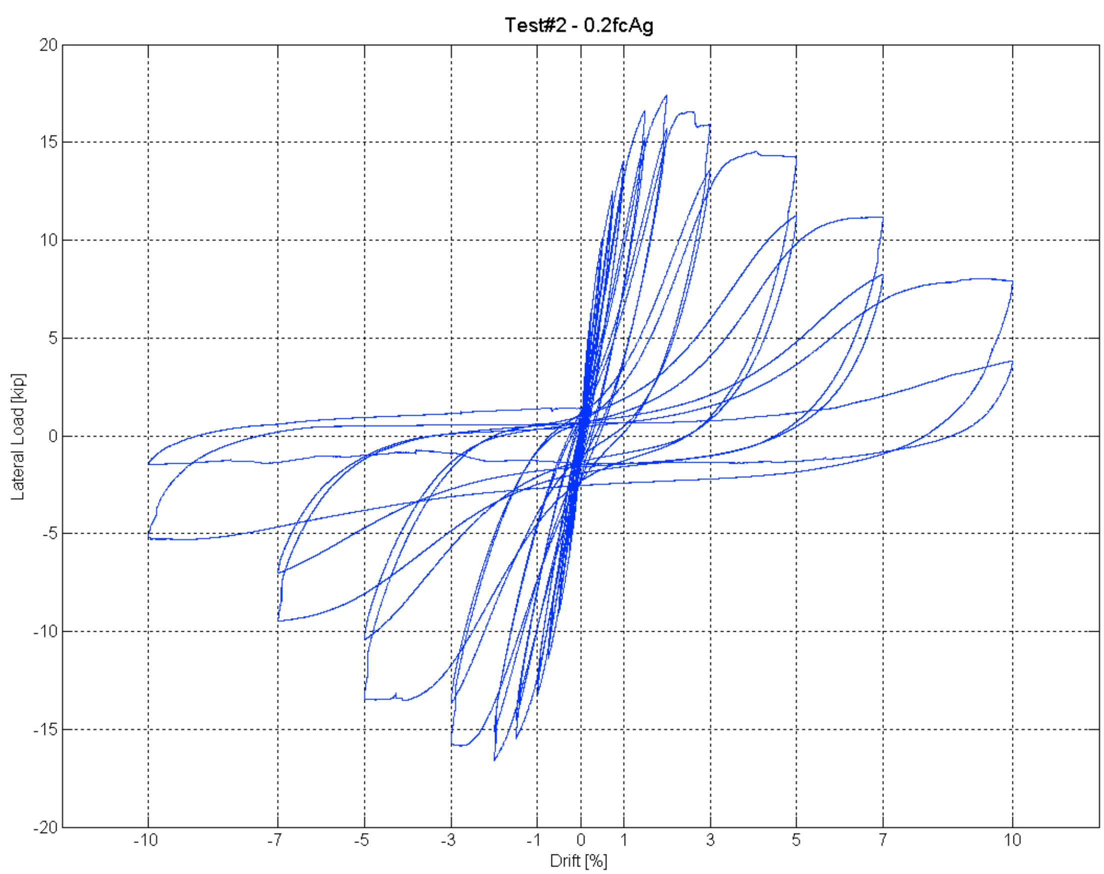

Figure 5.7: Exterior joint Force vs. Drift by M. Barnes et al.

It seems that the exterior joint is more resistant to displacements as indicated by the higher beam applied force. Although the strength for the exterior joint is initially higher and the stiffness greater, both the strength and stiffness degrade faster and eventually, at 10% drift, becomes weaker than the corner joint. Figure 5.8 shows the envelop curve for the corner joint.

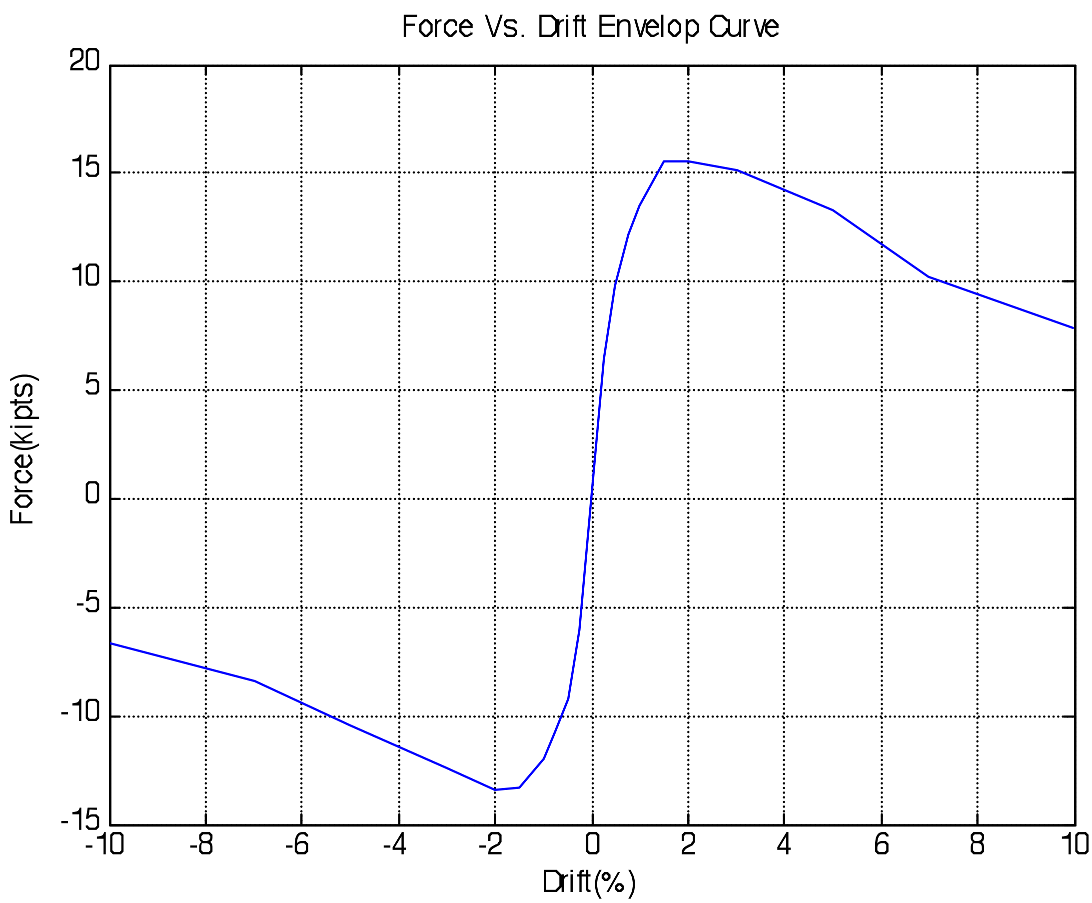

Figure 5.8: Corner joint Envelop Curve

Figure 5.8 shows a tabulated comparison of the results from M. Barnes exterior joint and the corner joint from this research. Both specimens were subjected to a constant compressive load of 20% of their compression capacity. The exterior joint yielded at a transverse load of 17.5

kips, while the corner joint took a transverse load of 15.5 kips. The exterior joint shear capacity was 1.358 kips, while the corner joint shear capacity was 1.203 kips. The joint shear stress normalized by the square root of f'c are 19 and 16 for exterior and corner joints respectively.

Specimen	**Joint Type**	**Column Axial Load**	**Max transverse load Vby (kips)**	**Joint shear at yield Vjy (kips)**	**Gamma (ɣ), shear stress normalized by the root of f'c.**
M. Barnes et al. 2008	Exterior	.2AgF'c	17.5	1.358	19
09-specimen	**Corner**	**.2AgF'c**	**15.5**	**1.203**	**16**

Table 5.9: Exterior vs. Corner joints comparison table

The fact that the corner joint only had one transverse stub while the exterior joint had two, there was more confinement in the exterior joint than the corner joint making the corner joint weaker as proven by the acquired results.

6. CONCLUSION

This research was conducted to assess the strength of RC building corner joints built prior to the adaption of modern seismic codes. The specimen tested was a half-scale model of a typical older beam-column corner joint. The following are deficiencies that the specimen was subjected to:

- The beam's flexural strength was greater than the column's.
- The hooks in the column were bent at 90 degrees.
- No transverse joint reinforcement.

The results acquired from this experiment show that the primary mode of failure is in the joint with significant spalling in the exterior face located behind the transverse beam. After comparing the results with M. Barnes et al. exterior joint results, it was found that the corner joint was weaker than the exterior joint.

Additionally, these results demonstrate that corner joints of buildings built prior to the adoption of the new design code are susceptible to failure if exposed to seismic action. Thus, causing total structural collapse. Therefore, rehabilitation of older non-ductile reinforce concrete buildings is necessary.

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