CHAPTER 8

CONCLUSIONS AND RECOMMENDATION

Countries in South East Asian region such as Thailand, Singapore and Malaysia were usually believed to be safe from seismic disaster. Several recent earthquakes have raised public concerns in seismic vulnerability of buildings in these countries. This research has put more attention on safety of the buildings. In the past, most buildings were not designed to resist earthquake. Therefore, these buildings are substandard with respect to modern seismic design code. In Thailand, almost buildings designed and constructed have some initial deficiencies as follows; i) stirrups in beam and column are usually widely spaced and the end hooks are non-seismic, ii) the column longitudinal bars are usually spliced immediately above the floor level and particularly, iii) no stirrups are provided in the joint. On these reasons, the representatives of these buildings are conducted and tested in order to get the conclusion as follows.

8.1 Conclusion

From the experimental program, substandard beam-column joint was conducted. The experiment aimed to elucidate the effect of various non-seismic details on the cyclic performance of beam-column joint. Particularly, the effects of anchorage bond, horizontal joint reinforcement and joint shear stress were investigated. Within the scope of the study, some preliminary conclusions and findings are listed as follows.

1. According to the conventional design and construction, some deficiencies of reinforcement details of conventional buildings do not meet the seismic provision standard. The lap splices of column bars just above joint are not the weaken point of all specimens since all columns can perform well with no or little splitting cracks observed at the spliced position. The little or no horizontal transverse hoop within joint core is one of the deficiencies which cause severe damages on the joint, especially joint of low and medium column tributary area.

2. From the relationship between the structural index of h_c/d_b ratio and the global anchorage bond, bond deterioration within joint region was not found in the representation of small, medium and large column tributary area (specimens JS, JM and JL) through test, corresponding to their h_c/d_b ratios passing ACI requirement(h_c/d_b ratio of 20). It may be concluded that although most of buildings are designed according to ACI with no seismic consideration, these building are safe from bond deterioration within joint core.

3. Representatives of medium and small column tributary area of conventional substandard buildings (specimen JM, JS) with less anchorage bond(h_c/d_b ratio of 29, and 25, respectively) do not lose bond. This conveys that joints of these specimens are demolished by only shear force not bond effect. Therefore, the strength of specimen was determined from joint strength not beam or column strength. In other words, the full capacity of beam and column has not been developed so the performance of specimen cannot be achieved as it should be.

4. Beam-column joint can meet the h_c/d_b ratio(33) requirement for ductile frame of NZS standard code, which reflects in term of column size. This joint of conventional building shows good seismic performance-spindle hysteresis shape and ductile flexural mode(ductility factor of 4.33) though it was designed for gravity load only and without ductile reinforcing detail. Only the representative of existing building with large column tributary area achieves good seismic hysteresis behavior.

5. The loss of spalled concrete section of specimen with complete unbonded bar(Specimen JB) is caused from large plastic strain of beam bar. The splitting cracks of concrete cover affect the reduction of member's capacity since the compressive zone of section decreases in accordance with the reduction of moment arm. This will affect the strength of frame. The reduction of strength in bondless joint frame is caused from the reduction of beam's capacity, which is different from the joint collapse of the conventional frame.

6. Specimen JB whose longitudinal beam bar bonding is removed initially can avoid joint shear failure. By this phenomenon, the concrete cover of beam is torn off and the compressive stress block of concrete moves down. The compression resultant on the opposite faces is then aligned with a flatter inclination. The capacity of compression strut is increased, which can reduce joint damage. Non-failure behaviors of the joint, the beam and column are able to develop their probable moment capacity. This result is useful in both of the structure's ductility and the retrofitting of buildings when encountered severe earthquake.

7. Only the provision of the substantial transverse hoops whether rectangular hoops (JR and JT) or spiral hoops (JA) within joint core can not prevent joint shear failure even though the bond anchorage is not lost. It may be stated that the ACI minimum joint shear reinforcement may not be adequate to avoid joint shear failure.

8. Utilization of the substantial amount of discrete confinement can slightly supplement the strength of joint but cannot enhance the post peak response. Not only it cannot only keep joint panel, but also help the beam-column joint performing good seismic behavior- narrow pinching, small energy dissipation of hysteresis loop. It is not enough to behave as fully ductile frame.

9. The provision of continuous confinement (Specimen JP) can obviously improve seismic performance after post peak such as full ductility(4.33%) and higher lateral strength(20%). However, it does not make better hysteresis loop in the aspects of narrow pinching and small energy dissipation since a number of localized cracks are concentrated on the beam-joint interface only.

10. The development of tensile strain in steel bars was related to the failure mode rather than dependent on bond anchorage in joint core. When the failure occurred in beam, high tensile strain was developed in steel but when the failure occurred in joint, the strain in steel was reduced. The reinforcing bar could develop high plastic strain even without bond in the joint since anchorage zone can be found along the bar outside the joint region.

11. The local pull-out crack is not related to the h_c/d_b ratio and the amount of horizontal joint reinforcement since specimens with either small or large tributary area or specimens with modified joint reinforcement cannot avoid the pull-out crack at the column face.

12. Hysteresis loops of all modified specimens show pinching characteristic with low energy dissipation because a number of localized cracks are found at any positions such as at the corner of joint face due to crushing of concrete stress block and at the middle of joint core due to opening, closing and sliding of cracks in the joint. Hysteresis loops of these specimens are quite different in the case of specimen with large column tributary area since its hysteresis loop shows the spindle-shaped with large energy dissipation.

13. The hysteretic behavior of most specimens modified with various joint reinforcements (specimen JD, JR, JT, JA and JP) can attain higher strength (15-20%) than that of the control specimen. Failure mode of specimens JR, JT and JA can be regarded as joint brittle failure while failure mode of specimen JD can be considered as combined joint shear and crushing failure. The failure mode of specimen JP is classified as ductile failure. Only specimen JP can pass the criteria of fully ductile frame according to NZS standard code (ductility factor of 3.0) while other specimens can only pass the criteria of limited ductile frame (ductility factor of 1.25).

14. The proposed numerical model of interior beam-column joint is considered for significant effects of rectangular and spiral joint reinforcement, splitting cracks of concrete covering, bond deterioration including the capacity of adjoining circumstance members (beam and column). This model is satisfactorily proved to be used as rational tool for the evaluation of existing beam-column frame and the design.

15. From the numerical investigation, conventional buildings without seismic consideration are mostly failed under joint shear force. They can be improved their seismic performance by expansion of column width (b_c). This method is fairly achieved for the oblong column buildings without loss bond anchorage. If the column dimension of the buildings is not typically oblong, this expansion of column width (b_c) may not be archived due to effect of bond deterioration.

8.2 Recommendation

From the previous experimental results, the conventional buildings designed in accordance with ACI standard code with gravity load only are considered as not adequate to resist the severe seismic attacks since their seismic behaviors- narrow and pinched are poor. Moreover, the failure mode is quite brittle and the peak strength at relative drift ratio do not satisfy the acceptance criteria of ATC-40 report (2.0% drift). In this research, the beam-column joint is modified to improve and enhance the performances of conventional buildings. The interesting discoveries are described as follows:

1. Since the size of column has an influence on the seismic performances, therefore, the beam-column joint in substandard existing frame may perform in a ductile flexural mode. The research is needed to investigate the effective size and

stress condition of column that can serve as prioritize substandard RC buildings for seismic rehabilitation.

2. Within the scope of the test program, the complete unbonded beam bar within the joint was found to be beneficial in the occurrence of moderate earthquake, and is very helpful in terms of post earthquake repair because of no joint failure. In this regards, the unbonding can perhaps be considered in the new constructions. Further research is needed to clarify the behavior of interior and exterior beam-column joint with unbonded bars.

3. The usage of transverse joint reinforcement was not successful in the viewpoints of seismic performance. One of problem is a spacing of transverse confinement according to ACI standard code. Since the substantial joint reinforcements do not help to prevent joint failure, thus full ductile behavior does not occur. Therefore, spacing of joint confinement should be tested and studied more if it is still considered as the best way to overcome the moderate to severe earthquake loads.

4. The continuous joint confinement should be recommended rather than the discrete joint confinement because it can enhance the effective seismic performance of the joint against the moderately seismic action. Besides, it can not be applied easily in the construction process. However, some hysteresis behaviors such as pinching characteristic are needed to be considered further more.

5. In terms of increasing construction cost, the amount of continuous joint confinement should be reconsidered because the joint reinforcements did not yet develop its full capacity. Therefore, the stress and strain of joint reinforcement corresponding to shear force should be the indicators of the volume of reinforcing steel within joint core.

6. In views of the significance of the problem, further research works related to anchorage bond, joint shear stress and horizontal joint reinforcements are needed to strengthen the above conclusions and to realize the implementation of the findings in this paper. The condition of existing stress and strain due to gravity load on the beam should also be studied.

7. The author's model should be modified with various special joint reinforcements such as "Z-shape" or improved for limitation of substantial transverse joint reinforcement due to proper amount of that still questioned.

8. For all practical purpose, the author's model should be further developed with three-dimension building frame because transverse adjoining beam affects on seismic performance as ACI 's provision. Furthermore, this model should be adapted to assess seismic performance of retrofitted buildings whether at beam, column, or joint failure.