Chapter 6

Conclusions and Recommendations

6.1 Conclusions from experimental studies

The two interior Rib beam-column subassemblages were tested. The first test specimen showed inadequate performance under lateral cyclic loading. The second test specimen, after the CFRP rectification, behaved very well compared to the first specimen. The second specimen showed no sign of overall strength degradation even up to a 4.0% drift ratio.

The deficiencies observed in the first test specimen, which was detailed according to current non-seismic detailing requirements specified in Australian codes (AS-1170.4, 1993; AS-3600, 2001) were as follows:

- The inadequate length of the main top reinforcement has led to sever cracking of flange slab, which was detrimental to the overall performance. Hence this type of crack development should be avoided as it might trigger a flexure-shear failure mechanism and lead to catastrophic failure.
- Excessive yielding and slippage of the bottom reinforcement at beam column interface.
- Slight crushing of concrete at bottom of rib beam-column interface, indicating that rib width is inadequate to transfer compressive force at higher drift levels.

The most severe of above deficiencies were considered to be the occurrence of a wide flexural crack at the curtailment point of the main top reinforcement. This particular crack initiated at a drift ratio of approximately 2% and grew to a width in the order of 5mm at a drift ratio of 3.0%. Such a wide cracks are of concern as they are associated with very large local strains in mesh reinforcement in the flange slab, which could result in fracture, and may lead to even complete failure of the member.

The second test was performed with the CFRP retrofitted specimen. The CFRP repair was done to avoid failure observed in the first test. The inadequate length of top bar was properly addressed in the second test specimen by continuing CFRP strips, 600 mm beyond the first curtailment point.

The second specimen performed very well under test conditions, with a higher ultimate strength than achieved for the first specimen. It should be noted that much better performance could be expected if the improved reinforcement detailing was used in a new subassembly. However, following conclusions can be made from the second test:

- Second test has demonstrated the effectiveness of CFRP as viable repair/strengthening system. The technique used here could be used to rectify existing structures with detailing deficiencies.
- Reference to the time-history analyses, the revised detailing is suitable to withstand very large earthquakes without significant structural damage.

6.2 Finite element analysis

Finite element analysis was conducted and reported in chapter 5 to investigate the performance of ribbed beam-column connection. The first finite element analysis model (FEM) was developed to compare the experimental results of the first test subassemblage. Second finite element model results were used to evaluate the performance of the improved detailing used. The only modification made was extending the length of main top reinforcement bar to 1600 mm from 1000 mm. The shear transfer coefficient was calibrated with the first finite element model. Once it is calibrated finite element modelling procedure can be used to obtain more information compared to conventional type laboratory test.

6.3 Design recommendations

A comprehensive study of the seismic behaviour of reinforced concrete flexural members generally requires a wide-range of investigation. However, a significant understanding of the behaviour may be obtained from a relatively small investigation, the design recommendations are developed from the limited test data and analytical results. It is with this belief that the present investigation was carried out. The recommendations are drawn as follows:

- The design of rib beam for negative and positive bending moment shall be carried out in the similar way to the calculations presented in Appendix-A, which is mainly based on the Australian code (AS3600, 2001) method to the normal T-beams.
- The effective flange width for flexural calculations and stiffness calculations shall be taken as shown in figure 2.2(Paulay et al., 1992) of chapter 2.
- The negative reinforcement requirement over the supports shall be determined ignoring the slab reinforcement for the gravity load case (1.2 G+ 1.5 Q), the total area of reinforcement to be considered for the earthquake load combination case (1.0G+0.4Q+EQ).

- The top reinforcement shall be curtailed as per the Australian code deemed to satisfy requirement (i.e. As per clause 8.1.8.6 of AS 3600). If the structure or loading is not satisfying the requirements of the above clause, curtailment shall be checked for theoretical bending moment diagram and provide the curtailment length for the greater length.
- Shear links shall be provided as per the Australian Code (AS-3600, 2001) requirements for beams. Further investigation is required to consider 10 % shear enhancement provided in other codes. The shape of shear link shall be similar to type-3 (see Figure 2-14 of chapter 2) with open top.

6.4 Recommendations for further work

Further testing and analytical work is required to investigate the shear behaviour of the rib slab system. The minimum rib width required to prevent crushing near the beam column interface is needed to be studied. Only one beam size was used in testing. Further tests should be conducted with different beam sizes to confirm the observations reported in this thesis.

6.4.1 Influence of flange slab reinforcement

The main influence of the slab on the inelastic behaviour of flange-beams was the contribution of slab reinforcement to the top tensile steel area. This was discussed in chapter 2. A Similar behaviour was observed in rib beam-column subassemblage testing. However, it should be noted that the negative reinforcement, which is the normal practice in gravity load design. The finite element analysis shows clearly the slab mesh contribution even at low drift levels. It was also seen that slab reinforcement stress variation across the width was marginal. This effect will increase the downward (negative) moment capacity due to slab reinforcement, and cause more energy dissipation per cycle. However, this increase imposed higher compression in the bottom compression zone, and higher shear force acting in the downward direction. These increased compression and shear forces could cause early buckling of bottom bars and increase the amount of shear degradation. These factors should be considered in the analysis and design of the critical regions near beam-column connections.

6.4.2 Amount of bottom reinforcement

If full deformational reversals are expected to occur in the beam critical regions near the column connections, to improve energy dissipation capacity, it is recommended that the bottom (positive moment) steel to be at least 75 percent of the top (negative moment) steel (Shao-Yeh et al., 1976). However this recommendation has been given for T-beams with full moment reversal situations. In the test assemblage only 40 % of top steel area (Including slab steel area) was provided for the bottom steel as per the critical bending moment envelope. This issue needs further research for recommendation for the Corcon slab system.

6.4.3 Shear reinforcement

The design of shear links in rib beam was done as per the Australian Code (AS-3600, 2001) requirement. However, 10% shear enhancement provided in other codes (ACI-318, 1999; BS-8110, 1995; SANZ, 1982) was not considered as Corcon rib slab system was not according to the rib spacing limitation specified in the above codes. Buckling of bottom bars or deficiency in shear capacity of the beam was not seen during the testing. Therefore, shear link provision in Australian code(AS-3600, 2001) is sufficient and further reduction in shear links need to be studied.

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