10. Conclusions

The following conclusions are taken from the results presented within this thesis. Seventy reinforced concrete external beam-column connection specimens were tested and over three hundred finite element modelling runs took place.

The conclusions are split into three main sections :

10.1 Conclusions from the Monotonic Test Programme
10.2 Conclusions from the Finite Element Analysis
10.3 Conclusions from the Cyclic Test Programme

Joint shear stresses are referred to throughout these conclusions. These were calculated using the methods outlined previously in Section 1.3. The influence of concrete strength on the joint shear stress was *normalised* by dividing the value of joint shear stress by the square root of the compressive cylinder strength.

10.1 <u>Conclusions from the Monotonic Test Programme</u>

10.1.1 INITIAL JOINT CRACKING CAPACITY

10.1.1.1 The value of normalised shear stress at initial joint cracking was not influenced by the quantity or positioning of joint ties.

10.1.1.2 The value of normalised shear stress at initial joint cracking was not influenced by the use of high strength concrete.

10.1.1.3 The value of normalised shear stress at initial joint cracking was not influenced by the joint aspect ratio.

10.1.1.4 The author recommends the use of BS 8110 equation 6b [3] when it is necessary to avoid shear cracking prior to the ultimate limit state :

$$v_c' = v_c \sqrt{\left(1 + \frac{N}{b_c h_c v_c}\right)} \quad \text{........(eq. 10.1.1.4 / BS 8110 6b)}$$

where v_c is the design concrete shear stress for a reinforced concrete section (BS 8110 Table 3.9),

N is the column load,

b_c and h_c are the column breadth and height respectively.

10.1.1.1 Increasing the joint capacity at initial cracking

10.1.1.1.1 The value of shear stress at initial joint cracking was increased by the use of fibre reinforced concrete. Using 2.3% of short or long fibres (as defined previously in Section 2.2.2) gave an enhancement of around 50% to the joint capacity.

10.1.1.1.2 The value of shear stress at initial joint cracking was increased by the use of steel shear plates within the joint. The use of a 4 mm steel deformed plate (as defined previously in Section 2.1) gave an enhancement of around 25% to the joint capacity.

10.1.2 ULTIMATE JOINT CAPACITY

10.1.2.1 The ultimate shear capacity of a joint, without shear reinforcement, under a *low column load**, was found to be directly proportional to the square root of its compressive cylinder strength.

$$v_{ult} = \sqrt{f_{ck}} \quad \text{........(eq. 10.1.2.1)}$$

where the compressive cylinder strength is related to the compressive cube strength by the following expression, $f_{ck} = 0.8 \times f_{cu}$ [17],

* the *low column load* within this investigation produced an axial stress in the column of 2.2 MPa or less.

This expression was also valid for high strength concrete (up to f_{ck} = 100 MPa).

10.1.2.2 The use of U-bar beam steel anchorage was found to reduce the ultimate shear capacity by a value of up to 17%. This was due to the U-bar transferring all of the beam's load into the joint region. The maximum bond stress at failure was located around the top bend of the U-bar. This value was an average of around 8 MPa for normal strength concrete. An average bond stress of over 12 MPa was achieved over this same region for high strength specimens.

10.1.2.3 Using bent down steel detail for the beam tension steel allowed the full capacity of the joint to be reached. The anchor leg transferred a large proportion of the beam's load into the lower column region. This value of bond stress at failure was significantly lower than for the U-bar steel at around 3 MPa.

10.1.2.4 The experimental specimens with a joint aspect ratio of 2.0 failed at a shear stress 15% lower than the value predicted by equation 10.1.2.1. This was believed to be due to slenderness effects within the joint core.

10.1.2.5 The two reduction effects highlighted in 10.1.2.2 and 10.1.2.4 should be incorporated into equation 10.1.2.1. This gives the following empirical expression for the evaluation of the ultimate joint capacity.

$$v_{ult} = \alpha\beta\sqrt{f_{ck}} \quad \text{........(eq. 10.1.2.4)}$$

where α is a reduction factor, $\alpha = 1$ for bent down beam steel,

$\alpha = 0.83$ for U-bar beam steel,

β is a reduction factor, $\beta = 0.25\left(5.4 - \dfrac{h_b}{h_c}\right)$,

h_b is the total depth of the beam,

h_c is the total depth of the column,

for specimens where $1.4 < h_b/h_c < 2.0$.

10.1.2.1 Increasing the ultimate joint capacity

10.1.2.1.1 The ultimate joint capacity was significantly enhanced through the use of joint ties. The placement of joint ties around the centre of the joint was found to increase the shear capacity of the concrete.

10.1.2.1.2 The placement of joint ties around the level of beam tension steel reduced the risk of anchorage induced joint failure by giving confinement to the concrete beneath the top bend of this rebar.

10.1.2.1.3 Within this investigation the placement of a single tie ($Aj/d_cb_c = 0.4\%$), in the top half of the joint below the level of beam tension steel, gave a joint enhancement of around 25%. The placement of three ties ($Aj/d_cb_c = 1.1\%$), in the top half of the joint below the level of beam tension steel, gave a joint enhancement of around 50%.

(where Aj is the joint steel cross sectional area,
d_c is the depth of the column and b_c is the width of the column).

10.1.2.1.4 Using joint ties which were not fully anchored resulted in a reduction in joint shear capacity of 10% due to slippage of the tie legs. To allow the full capacity of the joint to develop it was necessary to bend the free ends of the tie legs through at least 135°.

10.1.2.1.5 The ultimate joint capacity was significantly increased through the use of fibre reinforced concrete. Using 1.5% of short or long fibres (as defined previously in Section 2.2.2) gave an enhancement of over 40% to the joint capacity. Using 2.3% of short or long fibres gave an enhancement of over 65% to the joint capacity.

10.1.2.1.6 The ultimate joint capacity was significantly increased through the use of steel shear plates. Using a 4 mm steel deformed plate (as defined previously in Section 2.1) within a U-bar beam steel arrangement gave an enhancement of around 100% to the joint capacity.

10.2 Conclusions from the Finite Element Analysis

10.2.1 General modelling

10.2.1.1 A standard finite element mesh was proposed for use in beam-column connection design. Guidance was also proposed for the generation of the concrete and steel properties. Both normal and high strength concrete (as defined previously in Section 2.2.2) were considered. Guidance was also given on the placement of the steel layout for optimum performance.

10.2.1.2 The viability of both the finite element package and the mesh refinement technique was demonstrated by the successful modelling of simple problems with known solutions.

10.2.1.3 The standard finite element mesh design was shown to demonstrate four different failure mechanisms : flexural failure of both the beam and the column; and joint failure due to both shear effects and also those induced by beam anchorage slippage.

10.2.1.4 The eight standard, normal strength specimens from the experimental test series were modelled. The modelled failure loads showed excellent correlation with the experimental results. The mean average of the accuracy ($failure_{model}/failure_{test}$) was 1.01 and the variation coefficient was 8.1%.

10.2.1.5 The eight standard, high strength, specimens from the experimental test series were modelled. The modelled failure loads showed excellent correlation with the experimental results. The mean average of the accuracy was 0.99 and the variation coefficient was 4.3%.

10.2.1.6 Strain values from the reinforcing bars within the experimental specimens were modelled for both the normal and high strength specimens. The correlation was excellent for the main column and beam steel.

10.2.1.7 The modelling of the tie strains and the beam steel anchorage approaching failure was less accurate. This was due to the extensive crushing and cracking of the concrete elements within the joint.

10.2.1.8 The eight specimens investigating joint aspect ratio from the experimental test series were modelled. The modelled failure loads showed good correlation with the experimental results. The mean average of the accuracy was 0.88 and the variation coefficient was 7.1%.

10.2.1.9 Load-deflection comparisons were generally good for all of the modelled test series. However, the modelled response after initial joint cracking was slightly stiff.

10.2.1.10 The standard mesh design gave a good *lower bound* prediction for the initial joint cracking capacity for all of the modelled test series.

10.2.1.11 The same mesh design was adapted to model the seven test specimens by Reys de Ortiz [10] and the twelve test specimens by Parker [11]. Both modelled failure loads showed good correlation with the experimental results. For Reys de Ortiz's specimens the mean average of the accuracy was 0.90 and the variation coefficient was 7.5%. For Parker's specimens the mean average of the accuracy was 1.15 and the variation coefficient was 8.4%.

10.2.2 Parametric study

10.2.2.1 The influence of concrete strength on a beam-column joint was considered using sixty models of three different sizes. The ultimate shear capacity of a joint was found to be equal to the square root of its compressive cylinder strength.

$$v_{ult} = \sqrt{f_{ck}} \quad \text{........(eq. 10.2.2.1)}$$

where the same conditions applied as previously with equation 10.1.2.1

10.2.2.2 The influence of column axial stress on a beam-column joint was considered using twenty seven models and three different scale factors. The ultimate shear capacity was found to be enhanced by around 40% when subjected to an axial stress of 20 MPa. Prior to a stress of 20 MPa this relationship was reasonably linear, but this enhancement rapidly reduced with any further increase in axial stress.

10.2.2.3 The influence of joint tie positioning was investigated using seven models with bent down steel detail and seven models with U-bar beam steel detail. The optimum position for joint ties was found to be in the upper two thirds of the joint below the level of the beam tension steel.

10.2.2.4 The influence of beam steel anchorage was investigated using thirty models with two different joint aspect ratios. The use of U-bar beam steel anchorage was found to reduce the ultimate shear capacity by an average value of 18%.

10.2.2.5 The influence of joint aspect ratio was investigated using the results of sixteen models with two different beam steel details. The shear stress at failure reduced by around 25% as the joint aspect ratio increased from 1.4 to 2.0.

10.2.2.6 These enhancement / reduction factors may be incorporated into equation 10.2.2.1. This gives the following empirical expression for the evaluation of the ultimate shear capacity.

$$v_{ult} = \alpha\beta\gamma\sqrt{f_{ck}} \quad \text{........(eq. 10.2.2.6)}$$

where α is a reduction factor, $\alpha = 1$, for bent down beam steel,
(*beam anchorage*) $\alpha = 0.82$, for U-bar beam steel,

β is a reduction factor, $\beta = 0.42\left(3.8 - \frac{h_b}{h_c}\right)$,

(*joint aspect ratio*) h_b is the total depth of the beam,
h_c is the total depth of the column,
for specimens where $1.4 < h_b/h_c < 2.0$,

γ is an enhancement factor $\gamma = 0.02(50 + f_c)$,

(*column axial stress*) f_c is column axial stress,
$f_c < 20$ MPa or $0.4f_{ck}$ whichever is lowest.

Note

The enhancement factor for column axial stress was taken from the results of the modelling study. There have been no experimental tests within this thesis investigating the influence of column axial stress. Doubts were expressed in the Literature Review (Section 1.4.3) over whether column axial stress has any influence over the ultimate joint capacity.

10.3 Conclusions from the Cyclic Test Programme

The following conclusions are based on the cyclic testing methods outlined previously in Section 7.2.1.

10.3.1 Joint failure is undesirable during cyclic loading. Specimens which exhibited joint failure rapidly lost their strength. This behaviour under actual earthquake motions may result in structural collapse.

10.3.2 Joint failure was accompanied by excessive shear cracking of the joint concrete. This resulted in a reduction in bond of the beam tension steel and thus a lower joint capacity. With each subsequent cycle this bond loss increased as the specimen rapidly lost its strength.

10.3.3 The presence of ties raised the ultimate shear capacity of a joint. However, once joint failure occurred the presence of ties did not influence the deterioration rate of its strength.

10.3.4 The use of high strength concrete raised the ultimate shear capacity of a joint. However, once failure occurred the high strength specimens lost their strength more rapidly than their corresponding normal strength specimens.

10.3.5 The 4 mm shear plate within a U-bar steel arrangement (as defined previously in Section 2.1.3) raised the ultimate shear capacity of specimen C6LNP4. However, once failure occurred the presence of this shear plate did not influence the deterioration rate of its strength.

10.3.6 The 4 mm shear plate within the welded steel / anchor plate arrangement (as defined previously in Section 2.1.3) significantly raised the ultimate shear capacity of specimen C6PLNP4.

10.3.7 Specimens which had joints strong enough to allow beam flexural failure to occur exhibited the best behaviour. The yielding of the beam steel allowed the joint to remain largely intact. This allowed the full bond to be developed within the joint for each cycle.

10.3.8 Specimens were observed which failed initially due to beam flexure but then after subsequent cycles failed in the joint due to shear. Once joint failure occurred these specimens lost their strength rapidly.

10.3.9 Specimens designed to initiate beam flexural failure away from the joint face (as outlined previously in Section 7.2.2.4) had reduced deterioration of the joint during cycling. Within this investigation a specimen with a haunched beam was manufactured and tested and this was successful in keeping the joint intact. A specimen with additional beam steel within the joint, terminated 100 mm from the column face, was also manufactured and tested and this was successful in keeping the joint intact.

10.3.10 Structures which may have to resist earthquake motions, very high winds or blast effects should be designed to provide a joint shear strength substantially higher than the beam's flexural strength. Considerations regarding initiating beam flexural failure away from the joint face should also be made.

10.3.11 Structures which may have to resist earthquake motions, very high winds or blast effects should have sufficient steel anchorage for large displacements of the beam in both directions. A beam-column connection may exhibit excellent behaviour when loaded in the downward direction but rapidly deteriorate if loaded in the upward direction. Straight compression bars do not provide an acceptable anchorage length for large displacements in the upwards direction.