DESIGN AND TESTING OF REINFORCED CONCRETE FRAMES INCORPORATING THE SLOTTED BEAM DETAIL

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This paper was a reviewed poster paper at the 2012 NZSEE Conference in Christchurch.

ABSTRACT

Shortcomings of modern seismic design in reinforced concrete have necessitated the development of new systems capable of addressing these issues. Able to be constructed using existing industry techniques, the slotted beam is one such practicable, economic solution. While earlier research by Au (2010) showed promising results for this system, it also highlighted issues with bond of beam reinforcement within interior joints and understanding of the joint shear mechanism. This paper explains and addresses these issues through a summary of the desktop and experimental research undertaken. The results were encouraging with 2 specimens successfully tested without bar slip and minimal beam elongation.

1 INTRODUCTION

Beam elongation and its associated issues have long been known to be critical shortcomings of modern reinforced concrete frame design. This elongation is a result of plastic hinge formation – a cornerstone of modern seismic design – and has several consequences. Firstly, the resulting floor activation can significantly increase the moment capacity of the beam/floor system. This increases column demand and, in the worst case, can result in undesirable soft-storey failure. Secondly, beam elongation can lead to a loss of both gravity support and diaphragm action within floors (Lindsay, 2004; Matthews, 2004). Examples of beam elongation were observed in the recent Christchurch earthquake and, while not as severe as the cases above, rendered a number of buildings irreparable – an issue of increasing importance to building owners and insurance companies. Design solutions to overcome this problem are clearly needed, and the slotted beam is one such solution. Typical detailing elements for a slotted beam are illustrated in Figure 1:

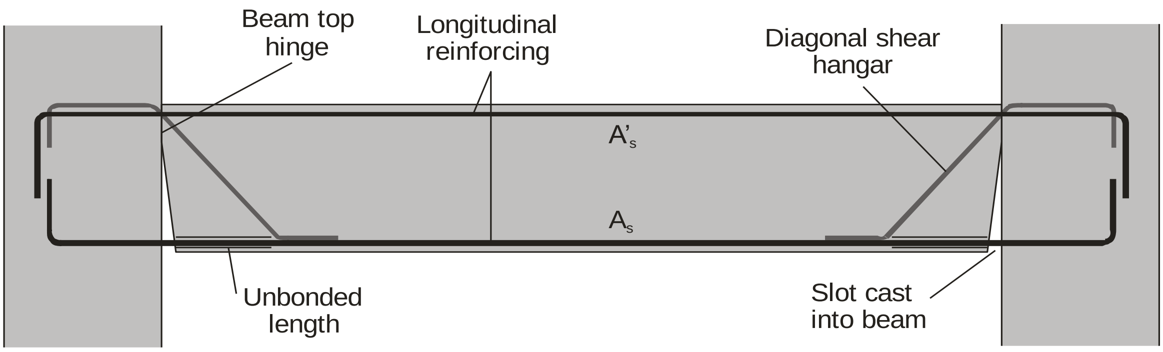

Figure 1: Diagram of a slotted reinforced concrete beam.

The research presented in this paper follows on from that of Au (2010) which highlighted several issues with current design provisions for slotted beams. Primary among these is the issue of bottom longitudinal beam reinforcement bond within interior column joints. Due to the presence of the slot, reinforcement is activated to overstrength on both sides of the joint simultaneously thus increasing the force transferred into the joint compared with a conventional reinforced concrete beam-column joint. Furthermore, with no concrete compression occurring at the beam soffit, the entire force must be transferred from the bottom bars into the joint via bond. If insufficient column depth is provided, bar slip can occur. Consequences of this include loss of energy dissipation capacity (ACI Committee 352, 2002), reduced system strength/stiffness, and reduction in available curvature ductility (Hakuto, Park & Tanaka, 1999). The correct beam-column joint shear mechanism – and thus the requirement for extra horizontal joint reinforcement – also needs to be established as the mechanism proposed by Au (2010) was based on a test in which significant bar slip occurred. Finally, current design procedure assumes the top longitudinal beam reinforcement remains nominally elastic and, therefore, seldom governs in terms of bond; the validity of this assumption needs to be confirmed. This paper outlines the desktop and experimental investigation undertaken in order to answer the above questions.

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2 DEVELOPMENT OF BOND EQUATIONS FOR SLOTTED BEAMS

2.1 Forensic Analysis of Au (2010) Specimens

Au (2010) tested 2 interior beam-column joint specimens with slotted beam details, one of which failed in bar slip of the bottom bars due to an insufficient column depth, hence insufficient bond. The key design oversights were assuming a reinforcement overstrength of 1.25 and adopting the same bond profile as used in conventional joints. Overstrength was subsequently shown to be on the order of 1.4 while the bond profile was unconservative due to increased reinforcement activation on both sides of the joint. The ratio of average bond stress, u_a, to maximum bond stress, u_{max}, was found to be 0.56 – compared with 0.67 for a conventional joint (Paulay & Priestley, 1992) – while effective column depth, h_c', was reduced from 0.8 to $0.75h_c$ due to localised pullout of joint concrete around the bottom beam reinforcement. Conversely, Au's (2010) second specimen was designed over conservatively resulting in a column depth more than double that required for an equivalent conventional frame. Given architectural and economic restraints, such a column depth would be unacceptable in current industry practice. However, review of this design revealed few oversights and the column depth could only be reduced by a small margin. From this forensic analysis, it became evident that some sort of bond improvement technique was necessary to reduce the required column depth to a level acceptable by industry.

2.2 Database Study

A study on approximately 20 beam-column specimens was undertaken to ascertain the key parameters affecting bond performance and to determine whether or not code provisions were suitable as a starting point. The most important correlation uncovered was that between increasing vertical joint reinforcement levels – generally provided by column bars – and maximum bond stresses achievable within the joint. This link is well documented (Eligehausen, Popov & Bertero, 1983; ACI Committee 408, 1992; Federation internationale du beton, 2000) and is due to the 'clamping' effect of reinforcement passing across the bond interface causing the bond mechanism to change from splitting to pullout. Splitting is an undesirable, brittle mechanism that occurs at low bond stresses and is generally associated with regions subject to poor confinement (Ogura, Bolander & Ichinose, 2008). It is driven by wedging action of the reinforcement lugs causing radial cracking around the bar and results in rapid failure. Pullout is a ductile mechanism associated with well confined concrete which changes the bond mechanism from bearing on the lugs to shear in the concrete between lugs (Tastani & Pantazopoulou, 2010).

Maximum bond stress within the joint, u_{max}, was found to be on the order of $2.5\sqrt{f_c'}$ for most specimens, consistent with the value assumed by Paulay and Priestley (1992) and on which the NZS3101:2006 recommendations are based. The provided column depth in each database specimen was then compared against code requirements and assessed for bar slip. NZS3101:2006 recommendations were found to be somewhat un-conservative for low values of axial stress with 20% of the specimens experiencing bar slip. Therefore, the slightly more conservative Paulay and Priestley (1992) recommendations were adopted as a basis for the slotted beam design equations.

2.3 Use of Supplementary Vertical Joint Stirrups for Bond Enhancement

In line with the findings of Sections 2.1 and 2.2, it was decided that supplementary vertical stirrups would be used within the joint region to enhance bond performance. These are more effective than column bars because they can be anchored within the joint such that they are not activated in flexure and thus remain elastic. Secondly, the stiffness of the response is greater because the stirrups can be mechanically anchored by end hooks around the beam bars – this reduces the extent of crack opening thus improving the bond performance. Furthermore, column bars only provide confinement to the beam bars located on the outside faces of the joint. Testing by Plizzari, Deldossi, and Massimo (1998) on monolithic pullout specimens indicated that use of these techniques could increase achievable bond stress by up to 45%. This figure was then reduced by 30% to allow for cyclic loading as opposed to monolithic based on a study by Viwathanatepa, Popov, and Bertero (1979). A further 25% reduction was applied to account for the fact that column bars have some influence on bond performance and the addition of vertical stirrups does not give cumulative gains in this situation. The resulting 20% performance increase was then applied on the tension side of the joint only as the compression side is already subject to highly effective confinement from axial stress. Combining this value with the Paulay and Priestley (1992) expression for column neutral axis depth, the bond modification factor for use of supplementary vertical stirrups, ξ_r, is established as given in Equation 1:

$$\xi_r = 1.15 - 0.17\frac{N^*}{A_g f_c'} \quad 1 < \xi_r < 1.15 \qquad (1)$$

Where N^* is the column axial load, A_g the gross column area, and f_c' the concrete compressive strength.

2.4 Bond Equations

The findings outlined above were then combined to form Equation 3 for required column depth for successful anchorage of bottom beam reinforcement:

$$u_a = 0.56u_{max} = 0.75\frac{h_c'}{h_c}x0.56x2.5\sqrt{f_c'} = \frac{d_b\lambda_o f_y}{2h_c} \qquad (2)$$

$$\therefore \frac{d_b}{h_c} \leq 2.1\frac{\xi_p\xi_r\xi_t\sqrt{f_c'}}{\lambda_o f_y} \qquad (1)$$

Where d_b is the beam reinforcement bar diameter, λ_o the system overstrength factor, f_y the reinforcement yield stress, and ξ_p and ξ_t as per NZS3101:2006. For the test specimens, this produced a column depth of 535 mm – approximately 1.7 times the depth required for an equivalent conventional column according to NZS3101:2006. Note that the provided column depth was 585 mm which allowed for disruption of bond due to strain gauges installed on the beam bars. For the top reinforcement, a slightly modified version of the NZS3101:2006 equation for nominally elastic reinforcement was used. The modification was to reflect a small reduction in the expected level of compression activation compared with an equivalent conventional frame. These requirements are given in Equation 4:

$$\frac{d_b}{h_c} = 3.6\frac{\sqrt{f_c'}}{f_y} \tag{4}$$

3 SLOTTED BEAM JOINT MECHANICS

3.1 Possible Joint Shear Mechanisms

Unlike in a conventional joint, there is no concrete compression from adjacent beam soffits. For this reason, Au (2010) proposed that a portion of the concrete strut force, ΔV_{sh}, must instead be resisted by additional horizontal joint stirrups within the bottom half of the joint as illustrated in Figure 2. The primary motivation for this assumption was that horizontal joint stirrups towards the bottom of the joint were activated more than those at the top during testing – similar to the profile shown in Figure 2. However, it is more likely that this asymmetric response was due to significant bar slip during the test. Once the bars began to slip through the joint, their stiffness was reduced significantly. With no compression from the adjacent beam soffit, the horizontal joint stirrups towards the bottom of the joint became the preferred mechanism of resistance and were thus activated to a greater extent than those at the top. Based on this observed response, Au (2010) recommended an additional 40% of the horizontal joint reinforcement required for an equivalent conventional joint be placed within the bottom half of the slotted joint. Such an increase is undesirable as it can lead to joint congestion and construction difficulties – especially when vertical stirrups are also used. In the proposed alternate mechanism, the concrete strut is realigned such that it is equilibrated by the relatively stiffer bottom beam bars. This requires that bar slip within the joint be prevented such that the stiffness hierarchy is not compromised. If this mechanism proves correct, the additional horizontal joint reinforcement recommended by Au (2010) would become unnecessary and could be excluded in future design.

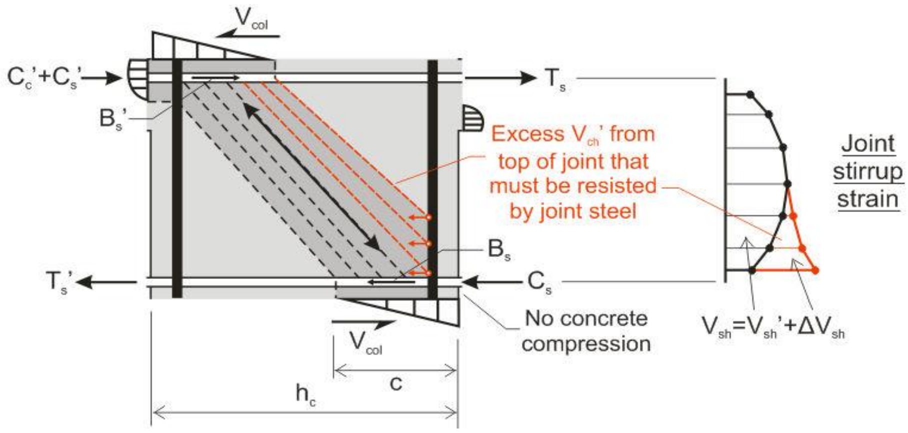

Figure 2: Interior slotted beam joint mechanism proposed by Au (2010).

3.2 Parametric Study

A parametric study on the relationship between various joint parameters and required horizontal joint shear reinforcement was carried out using a generalised version of the Paulay and Priestley (1992) equations. The parameters of interest included beam reinforcement asymmetry and overstrength factors, level of compression reinforcement activation, bond profile uniformity and effectiveness, column to beam height ratio, and axial stress. Applying upper and lower bound values for each of these variables, it was found that current NZS3101:2006 provisions are sufficient when the alternate mechanism discussed in Section 3.1 governs joint behaviour.

A second parametric study was then carried out to establish the sensitivity of ΔV_{sh} to these parameters. ΔV_{sh} is the portion of strut force resisted by additional horizontal joint reinforcement in the bottom of the joint due to a lack of compression from the beam soffit. This was undertaken to account for the possibility that the Au (2010) joint mechanism proved correct and additional joint reinforcement was required. When all variables were assigned their expected value, ΔV_{sh} was found to be 25% while use of upper and lower bound variables resulted in ΔV_{sh} of 35%. In practical terms, this translates to an additional 25 to 35% of horizontal joint reinforcement being required within the bottom half of the joint if the Au (2010) joint mechanism governs the behaviour. These values show good agreement with those of Au (2010) .

4 EXPERIMENTAL SETUP

4.1 Test Setup and Data Collection

Two beam-column specimens were constructed and tested at the University of Canterbury. These specimens were of the ‘cruciform’ configuration and on a scale of 80%. The specimens were tested under quasi-static cyclic loading according to ACI guidelines (ACI Committee 374, 2005) up to a lateral drift of 5.5%. This load was applied at the top of the column using a hydraulic ram and logged against column displacement measured by a rotary potentiometer attached to the other side of the column. Load cells attached to the ram and beam ends measured specimen reactions. The bulk of data was recorded by strain gauges attached at various locations on the reinforcement cage and linear potentiometers installed on the completed specimen. The experimental setup is shown in Figure 3.

4.2 Specimen and Material Properties

Reinforcement detailing was almost identical between specimens. A top to bottom beam reinforcement ratio of 2.06

was adopted in line with Au (2010) recommendations. This bias is required in order to reduce strains and thus damage in the beam top hinge. Unbonding tubes were mild steel while the shear hangers were dual HD16 bars – the higher grade necessary to keep the bar size down and provide prolonged elastic behaviour. Horizontal joint reinforcement in Specimen A was 135% $A_{jh,code}$ while B had the reduced value of 120%. Supplementary vertical joint stirrups were provided in both specimens as illustrated in Figure 4 using a combination of R10 and R12 bars to assess the effect of increased spacing on bond performance. Concrete strength at testing was 40 MPa for Specimen A and 30 MPa for Specimen B, although the design strength was for 40 MPa throughout.

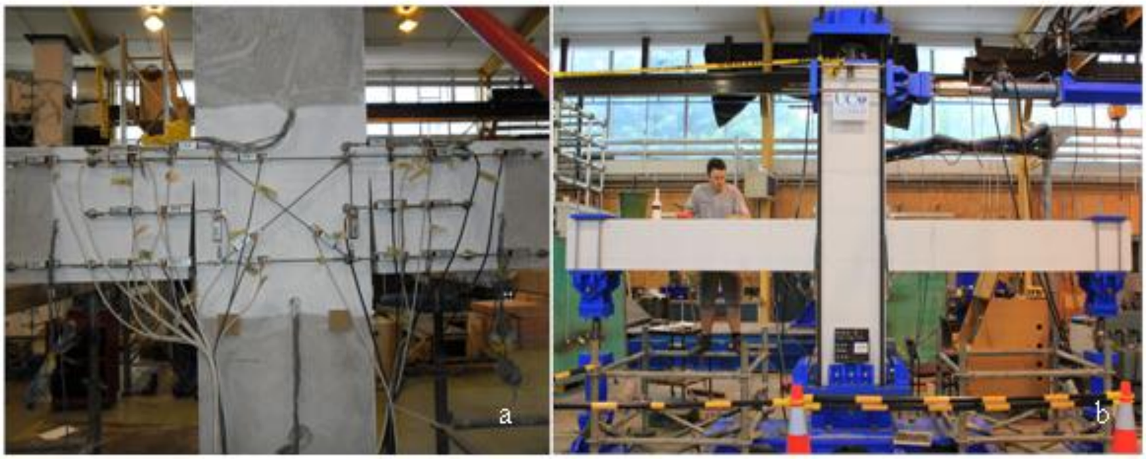

Figure 3: (a) Linear potentiometer layout; (b) Specimen in testing rig.

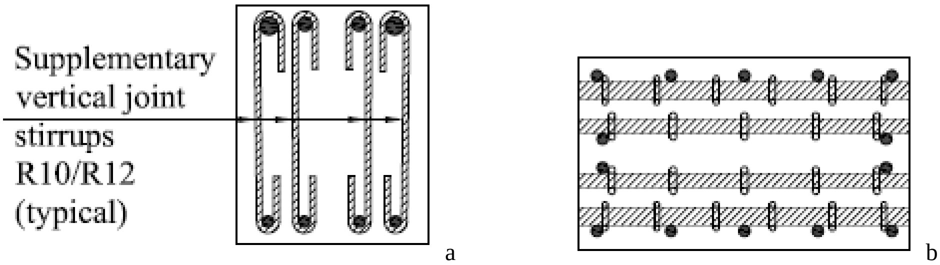

Figure 4: Supplementary vertical joint stirrups layout. (a) Beam section; (b) Column section.

5 EXPERIMENTAL RESULTS

5.1 Hysteretic Response and Observed Damage

The hysteretic response of each specimen is reproduced in Figure 5. Specimen A and B yielded at 0.65 and 0.67% drift, respectively, slightly greater than the predicted value of 0.57%. The corresponding yield force was 118 and 108 kN, respectively, showing good agreement with the predicted value of 113 kN from monolithic theory. Damage at this stage was limited to hairline flexural cracks in the beams and hinges in addition to minor cracks along the lines of the bottom beam bars that closed upon unloading.

Both specimens exhibited a ‘fat’ hysteretic response through 4.5% drift with no strength degradation. Indeed, post-yield stiffening is visible in cycles at higher drift levels. After the cycles at 4.5%, a single continuous crack approximately 4 mm wide was visible at the beam/column interface above the hinge on the gap closing side along with crushing and spalling damage on the beam top from previous compression cycles; this is shown in Figure 6a. On the gap opening side, a large crack of 3.5 mm maximum width had opened above the open slot as illustrated in Figure 6b. Concrete cone pullout could be seen around the bottom beam reinforcement where it entered the beam-column joint. Fortunately, the cone was confined to little more than the cover concrete as shown in Figure 6c, and it did not adversely affect the specimen response. Widespread flexural cracking reaching 0.35 mm width was visible in the beams in addition to minor cracks in the vicinity of the unbonded length. Steeply inclined joint cracks showed evidence of the strut still aligning to beam bars as proposed in Section 3.1; these can be clearly seen in Figure 6d. By this stage, strain penetration cracks had combined with joint shear cracks as shown in Figure 6e, but were still found to close during compression inducing cycles. Joint shear cracks reached a maximum width of 0.4 mm through the cycles at 4.5% drift. Evidence of bar buckling within the unbonding tubes during compression cycles was evident visually and by slight pinching in the hysteresis loops.

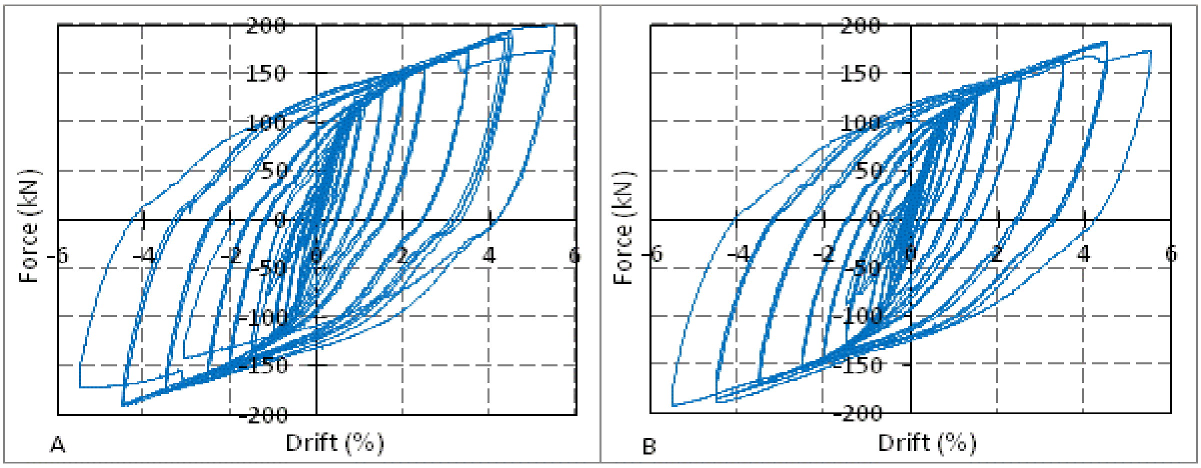

Figure 5: Lateral force vs. Inter-storey drift for Specimen A and B.

Testing continued to 5.5% drift where a single bottom bar fractured in tension in both specimens during the first cycle; an example of this can be seen in Figure 6f. While this resulted in a small loss of capacity, the general shape – and thus energy dissipation capacity – of the hysteresis loops was not compromised. Overstrength factors at 2.5%, corresponding to ULS drift limits, were found to be 1.32 and 1.40 for Specimens A and B, respectively. This shows good agreement with the figure of 1.35 adopted for design. Final overstrength factors were calculated to be 1.69 and 1.70 for Specimens A and B, respectively, indicating significant strain hardening of the reinforcement during cycles at larger drifts. However, this is above the theoretical upper bound obtained using the material overstrength factor of 1.5 found from reinforcement testing. Both tests were stopped midway through the 5.5% cycles on both tests due to fracture of a second bar and twisting of the beam.

5.1 Top Beam Reinforcement Activation and Beam Elongation

Top beam reinforcement in both specimens typically did not yield until 3.5% drift. This elastic behaviour contributed significantly to the minimal beam elongation observed – 2.8 and 3.2 mm at 4.5% drift for Specimen A and B, respectively. The geometric effect of simultaneous opening and closing gap rotations on either side of the joint – unique to slotted beams – is also responsible for the lack of beam elongation experienced. Combining the elongation over both beams gives a total elongation between columns of 0.7% beam depth, appreciably less than the 2 – 5% expected for conventional frames (Lau, 2001; Fenwick & Megget, 1993). The lack of top reinforcement activation and beam elongation observed through 4.5% drift indicates Grade 300 reinforcement is suitable in this application.

5.2 Bond Stress

No discernible bar slip occurred in either of the specimens. Average mobilised bond stress along the bottom bars was found to plateau at approximately 0.9 and $1.0\sqrt{f_c'}$ for Specimen A and B, respectively. Unfortunately, this may be inaccurate due to multiple strain gauge failures soon after yield. Back calculation using peak ram force at 4.5% drift suggested a value on the order of $1.35\sqrt{f_c'}$ to be more realistic. This is in agreement with the assumed value of $0.56u_{max}$ as discussed in Section 2.1. Despite the use of supplementary vertical stirrups, bond on the tension side of the joint was found to be 60% as effective as that on the compression side. This is to be expected. In light of the weaker than expected concrete strength on Specimen B, the effective column depth was reduced to 464 mm – approximately 1.5 times an equivalent conventional column. However, experimental evidence suggests column depth could be reduced further. According to the bottom reinforcement stress profiles within the joint, yield penetration did not reach more than 100 mm into the column from either side, while increases in the bottom reinforcement stress gradient within the joint were able to be sustained through 4.5% drift.

5.1 Joint Response

The joint response of both specimens was excellent with maximum stirrup activation on the order of ½ yield in Specimen B. While a reduced degree of activation was expected due to the increased joint shear area, this alone would not account for such a large reduction. It is thought that vertical stirrups contributing to the truss mechanism is responsible for some of this reduction, while some can be attributed to the larger column depth making the strut mechanism more effective. The latter mechanism was observed in the second test by Au (2010) where a relatively larger column was used, as compared to the code minimum. Although some bias in activation towards the bottom of the joint was visible in the response envelopes, this was not significant enough to cause concern. Furthermore, steeply inclined joint cracks were visible through the cycles at 4.5% as shown in Figure 6d. This indicates the beam bars were still able to provide an effective landing zone for the concrete strut and is further evidence of minimal bar slip. Based on these results it appears that current NZS3101:2006 equations for required joint shear area are more than adequate for joints adjacent to slotted beams.

Figure 6:** **Damage during 4.5% and 5.5% drift cycles (a) Crack above west hinge; (b) Full length crack through west hinge; (c) Cone pull out bottom west of joint; (d) Steeply inclined joint shear cracks; (e) Joint cracking and strain penetration; (f) Bar fracture.

6 CONCLUSIONS

The research presented above has answered several key questions currently preventing slotted beams from being used in NZ practise. Confirmation of their ability to reduce beam elongation and damage coupled with a stable response up to 5.5% drift shows they are a viable alternative to conventional frames. A workable solution to the issue of excessively large column depths was presented and tested successfully along with a simple equation for detailing this. Top beam reinforcement was confirmed to behave nominally elastic meaning existing NZS3101:2006 recommendations are applicable here. The viable joint shear mechanism was established and confirmed, with experimental results showing current NZS3101:2006 provisions are suitable for determining the required level of reinforcement. Perhaps one aspect of slotted beam behaviour requiring further research is that of bar fracture induced by low cycle fatigue. To determine the susceptibility to this failure mechanism, this should be tested with realistic seismic loading rates.

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