Magazine of Concrete Research, 2001, **53**, No. 01, February, 63–66

Discussion on paper published in *Magazine of Concrete Research* 1999, **51**, No. 6, 415–425

Strut and tie models for analysis/design of external beam-column joints

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The authors are complemented on an interesting paper, establishing an empirical method for designing external beam-column joints. The discussers and their research collaborators have recently sought to develop a rational design approach for beam-column bridge joints subjected to seismic loading,[1,2] and are well aware of the difficulties associated with producing simple design provisions as output from strut and tie modelling. Based on research conducted on exterior and interior bridge joints, the discussers offer the following comments in the hope that they may potentially improve or simplify the proposed methodology. Although several tests[3–7] conducted by the discussers cannot be used for verification of the proposed design approach, some conclusions from these studies can be applied to the analysis and design of external beam-column joints in frame structures.

Axial compression forces

The lower level columns of many concrete frames are often subjected to considerable axial compression due to dead and live loads. Similarly, prestressed sections are subjected to axial compression. Finally, cyclically-loaded frames can be expected to generate beam axial compression due to the cumulative effect of beam plastic hinge formation, and oscillating column seismic axial tension and compression forces. On this basis the discussers have found it valuable to always correctly account for axial forces at the joint boundaries when assessing likely joint performance.[1,2,6]

Initiation of joint cracking

The first indication of joint distress is the formation of shear cracks, and there would be some additional shear capacity in the joint beyond this state even when no shear reinforcement is provided in the joint. This is due to the participation of beam and column longitudinal reinforcement in joint force transfer. This is recognised in building codes (e.g. NZS 3101), where elastic column reinforcement placed between corner bars may be treated as directly analogous to vertical joint stirrups. Consequently it follows that a joint can be detailed with nominal reinforcement without any elaborate calculations if it can be assured that cracking of the joint is unlikely to develop under the factored design loads. In bridge joints, this was achieved by comparing the average joint principal tensile stress, which includes the axial load effects, to a predicted joint

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cracking strength of $3{\cdot}5\sqrt{f'_c}$, where f'_c is the joint concrete compressive strength. It is believed that a similar approach will simplify design of joints in most external beam-column joints.

D-regions of beams and columns

It is not surprising to note that the predicted forces did not agree well with those measured in the joint region based on the assumption that plane sections remain plane at the joint interfaces. As implied by the authors, the beam and column regions adjacent to the joint will be disturbed by the geometric discontinuity and thus they cannot be treated as B (or beam) regions. In order to obtain realistic estimates of the joint strut and tie forces, it is necessary to model the D (or disturbed) regions adjacent to the joint. The discussers have collected considerable strain gauge data from longitudinal reinforcement at the position of the member-joint interface. This data implies when member longitudinal reinforcing bars are anchored into the joint with hooks or U bars, the forces in the D-region of the member are less affected when compared to a member whose main reinforcement is anchored into the joint with straight bars. The disturbance in the D-region imposes additional demand on the longitudinal and transverse reinforcement of the member adjacent to the joint.[2,7] This is consistent with the finding by the authors in that the greatest discrepancies between the predicted and measured values were for the inner column tension force and the compressive force in the external column bars. By modelling the D-region of the upper column above the top joint interface, these forces can be more accurately predicted. In bridge joints, it was found both experimentally and analytically that the longitudinal beam bars can be subjected to up to 34% higher tension force than that predicted by the simple beam theory when the plastic moment is fully developed in the column adjacent to the joint interface as required by the seismic design philosophy.[7]

Detailing of hook reinforcement

The discussers have acquired experimental data confirming that for joints with little or no horizontal joint stirrups, the unrestrained hook of the embedded longitudinal beam tension reinforcement may rotate out of the joint about the diagonal compression strut, resulting in spalling of cover concrete on the back face of the joint. Even for well restrained hooks, it is unlikely that the cover concrete on the outside face of the joint can support significant flexural compression stress from the column above. Fuurthermore, flexural cracking at the upper column-joint interface and penetration of beam flexural cracks into the joint results in minimal bond being developed prior to the beam reinforcement hook. The resultant top column flexural compression force is reoriented within the member D-region adjacent to the joint as indicated above. Subsequently, the nodal dimensions in the upper left corner of the joint are primarily dictated by the bending radius of the reinforcement hook. Therefore, it is important to consider the bending radius of the hook reinforcement when establishing the joint strength. Also note that tests by the discussers have incorporated bending radii approaching 300% of code minima values to ensure satisfactory joint response. No comment on the influence of beam reinforcement bending radius on the joint strength was made in the paper.

Stiffness analysis

Incorporating the beam and column D-regions in the strut-and-tie model is also a concern for the stiffness analysis method discussed in the paper. If the D-regions adjacent to the joint faces are ignored, one can significantly underestimate the stresses in the joint struts.

Joint reinforcement

There are several issues surrounding the authors treatment of joint reinforcement. Firstly, there has been no consideration of vertical joint reinforcement typically represented by intermediate column bars, such that the resultant strut and tie model presented by the authors is not analogous to that forming the basis of the New Zealand approach.[8,9] Vertical joint reinforcement could be expected to at least partly aid in transmitting the beam flexural tension force to the diagonal joint strut, effectively broadening the joint strut width. In addition, the authors have assumed that only the horizontal stirrups in the top 62·5% of that region of the joint below the beam tension reinforcement are effective, although no reason for this assumption is given. Assuming that these upper stirrups are equally effective, the centroid of the horizontal tie force should be about 69% of the joint height, and should therefore be raised in the diagrams presented in the paper. The discussers also advocate that not all stirrups are equally effective, and have data to support this assumption. The discussers contend that horizontal stirrup effectiveness is largely dependent on the angle of bond struts necessary to shed force from vertical column reinforcement towards the CCC joint node, and that in this case the lower horizontal stirrups are relatively less effective.

Calibration

Calibration of the model with Ortiz's data, who used $f_{yb} = 720$ MPa, may not accurately capture behaviour for reinforcement strengths of about 500 MPa, as is typical in many countries. Indeed this yield strength was not approached by any of the other researchers considered in Table 1 of the paper, and would raise doubt about the suitability of assuming $\varepsilon_t = 0{\cdot}003$ in the formulation of equation (4).

Concrete tension carrying capacity

Analysis of several bridge joints confirmed that the tension capacity of cracked concrete significantly contributes to shear strength of reinforced concrete joints.[1,7] Although this quantity can be conservatively ignored in the design of joints, it plays a significant role when characterising behaviour of joints. Ignoring the concrete tension carrying capacity can underestimate the joint strength markedly, resulting in poor agreement with experimentally measured values.

Having made the above comments, the discussers again complement the authors on their paper, and are encouraged to see an independent study that has used a similar approach that which the discussers have adopted.

Reply by the authors

We thank the discussers for their kindly and informed discussion of our paper. We note that their research is concerned with joints subject to seismic loading. The authors believe that caution is required in extrapolating conclusions from cyclically loaded specimens to monotonically loaded specimens owing to differences in loading and reinforcement. We comment on the discussers points 1 to 8 as follows:

(1) *Axial compressive forces*—The effect of column load on joint shear strength depends on whether the loading is cyclic or monotonic. As discussed in the paper, the authors analysed data from tests[1] on monotonically loaded external beam joints and found no correlation between column load and joint shear strength. Further tests are required to confirm this.

(2) *Initiation of joint cracking*—We welcome the simplified approach proposed by the discussers but believe that it is too conservative for monotonically loaded joints. Restricting the average joint principal tensile stress to a joint cracking strength of $3.5\sqrt{f'_c}$ psi ($0.29\sqrt{f'_c}$ MPa) for joints with nominal shear reinforcement implies that joint shear strength depends significantly on column axial load which is not the case. Furthermore, the joint shear strength of specimens without joint shear reinforcement is significantly greater than the cracking strength (up to 3 times for the Oritz[2] tests) if the column load is low. We believe that our simplified design[1] method is more realistic and simpler to apply since the column load is not required

(3/5) *D regions*—We note the discussers points with interest and are pleased to see that their strain data accords with our findings. We agree that our model would be improved by modelling the D regions but at the expense of more complexity.

(4) *Detailing of hook reinforcement*—The effect of the radius of bend was not considered in the model since test data from monotonically loaded connections is inconclusive. For example, Oritz[2] varied the radius of bend between 4 and 8 bar diameters and found no significant effect on joint shear strength.

(6) *Joint reinforcement*—Vertical joint reinforcement was not considered in the authors model since the test data considered was for specimens without intermediate column bars. Further tests are required to assess the effect of intermediate column bars on monotonically loaded joints. The discussers comments on the position of the horizontal tie force in the figures of the paper are only correct if joint stirrups are equally spaced through the joint depth. This was the exception for the data considered. The authors agree with the discussers comments on horizontal stirrup effectiveness. In the model, stirrups are only assumed effective if placed above the flexural compressive zone in the beam which was assumed to be of constant depth.

(7) *Calibration*—A strain $\varepsilon_t = 0.003$ was adopted in the formulation of equation 4 since (1) stirrups typically yield at strains less than 0.003; and (2) the corresponding strain softened concrete strength is similar to that given by CEB Model Code 1990[3] for cracked concrete. The exact value adopted for ε_t is unimportant since the strut width was calibrated accordingly.

(8) *Concrete tension carrying capacity*—Whilst agreeing with the discussers, we note that our model indirectly accounts for the concrete tenison carrying capacity since it is calibrated to predict observed joint strengths. If the concrete tension capacity were modelled, the contribution of the direct strut would reduce.

Having made these comments, we again thank the discussers for their comments.

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