Seismic design criteria for reinforcement anchorages at interior R/C beam-column joints

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Abstract

The requirements for anchorage of beam longitudinal reinforcement at interior beam-column joints in earthquake resistant reinforced concrete moment resisting frames are re-evaluated. An introductory comparison of international design criteria shows that a broad disparity currently exists between these requirements. The suitability of existing criteria was assessed by assembling a database of over ninety tests of interior beam-column joint tests and comparing the assessed test performance with the performance that was predicted by each design criterion. This comparison showed that none of the existing design criteria was adequately able to predict experimental performance. In order to improve the design of earthquake resistant reinforced concrete frame structures an improved design criterion was developed by parametrically determining the influence of important variables on anchorage performance.

CE Database subject headings:

Seismic design; reinforced concrete; beam-column joints; anchorages; reinforcing steel; design codes

Introduction

Poor seismic performance of reinforced concrete moment resisting frames can be expected to occur if the beam-column joints in such structures are not appropriately designed and detailed. This expectation is due to the fact that joint failure can jeopardise the axial load capacity of

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the column framing into the joint, and can also reduce the stiffness of the structure. Referring specifically to interior reinforced concrete beam-column joints, two failure modes are typically considered. These two modes are joint shear failure and anchorage (or bond) failure. While there continues to be international disagreement about the design of beam-column joints (Park 2002), it is generally recognised that the consequences of joint shear failure are more severe than the consequences of bond failure in a joint, because shear failure has a brittle nature and is likely to compromise column axial capacity (Paulay and Priestley 1992). The main consequences of bond failure in a beam-column joint are that the stiffness and hysteretic energy dissipation of the joint are significantly reduced (Paulay 1988). A further motivation for designing to avoid bond failure during earthquakes is that damage due to bond failure is difficult to detect, and if detected is difficult and costly to repair. Hakuto et al. (1999) have also shown that severe bond failure can reduce the ductility capacity of the beams adjacent to a joint. Despite the lesser severity of bond failure it is nonetheless important to detail joints so that bond failure is avoided, particularly if the "capacity design" philosophy is being used to design the structure (Park and Paulay 1975).

Beam longitudinal reinforcement anchorage at interior beam-column joints

Anchorage of beam longitudinal reinforcement at interior beam-column joints generally relies on bond between the concrete and the straight length of reinforcing bar passing through the joint. Anchorage demands are particularly severe at interior beam-column joints due to the beam longitudinal reinforcement being in compression at one face of the joint and tension at the opposite face of the joint, as shown in Figure 1.

The free body diagram shown in Figure 1 is used as the basis of design criteria for anchorage of reinforcement at interior beam column joints. When the capacity design philosophy is used the magnitude of the tensile stress is assumed to equal the overstrength stress of the reinforcing bar (i.e. the maximum stress likely to occur in the reinforcement, including allowance for strain hardening and material property variation), and the magnitude of the compression stress is expected to be less than the overstrength stress of the reinforcing bar. Equilibrium of the reinforcing bar must be maintained, and in the absence of mechanical anchors, the equilibrating force is generated by bond stresses along the length of the bar passing through the joint. The distribution of these bond stresses is known to be complex, and to vary depending on the prior load history of the bar (Paulay and Priestley 1992), but for design purposes it is normally assumed that an average bond stress acts along the complete length of the reinforcing bar in the joint core. Applying the requirements of force equilibrium to the free body diagram of the bar it can be shown that the following inequality must be true if bond failure is to be prevented:

$$\pi d_b h_c \alpha_p u_b \geq \frac{\pi d_b^2}{4} \alpha_o f_y (1+\kappa) \qquad (1)$$

where d_b is the bar diameter of the beam longitudinal reinforcement, h_c is the column depth, u_b is the average bond stress available over the column depth, α_p is a variable included to account for the influence of column axial compression on bond strength, α_o and f_y are respectively the overstrength factor and yield strength of the beam longitudinal reinforcement, and $0 \leq \kappa \leq 1$ is a constant equal to the reinforcement compression stress divided by the reinforcement overstrength stress.

Equation 1 is the basis for many design criteria for interior joint anchorages; however three questions complicate definition of such criteria:

- What is the relationship, $u_b = f(f_c^{'})$, between concrete compressive strength and available bond strength?
- How is the available bond strength affected by column axial compression, i.e. what function defines α_p?
- What level of compression stress is likely to exist in the reinforcement, i.e. what function defines κ?

Comparison of existing international design criteria shows that considerable disagree currently exists with respect to all three of these questions.

<u>Comparison of current international design criteria</u>

There are currently at least four different design criteria used by different international concrete design codes to determine anchorage requirements for beam reinforcement at interior beam-column joints:

- The requirement of the U.S. concrete code (ACI 318 2008), which states that bar diameter not exceed 1/20 of column depth (i.e. $d_b/h_c \leq 1/20$).
- The design criterion included in the 1999 edition of the guidelines for designing earthquake resistant concrete buildings, published by the Architectural Institute of Japan (AIJ 1999).
- The design criterion included in the New Zealand Concrete Structures Standard (NZS 3101 2006).
- The design criterion included in the European design standard for earthquake resistant buildings (BS EN 1998-1).

All of the criteria listed above are based on the format given by equation 1, except for the U.S. design criterion. The functions used by each of these three criteria to define the relationships for u_b, α_p, and κ are shown in Table 1.

Unfortunately the different design criteria detailed above do not produce comparable outcomes. For a “reference” beam-column joint assumed to have equal areas of top and bottom beam longitudinal reinforcement, and with no significant column axial load, the considered anchorage criteria lead to the result shown in Figure 2, where the ratio of required column depth to bar diameter is plotted for various concrete strengths. Based on Figure 2 and the results of more detailed comparisons (Brooke 2011) of the various design criteria components listed in Table 1, the following observations can be made about the four design criteria:

- The least conservative design criterion is that of ACI 318, followed by that of NZS 3101.
- The EC8 design criterion relies on lower available bond stresses (in the absence of column axial compression) than either the AIJ or NZS 3101 criteria.
- The AIJ design criterion requires larger reinforcement compression stresses to be anchored than either the EC8 or NZS 3101 criteria.
- The EC8 and AIJ criteria permit a greater increase of reliable bond strength as column axial compression increases than does the NZS 3101 criterion.

Recognising the marked differences between criteria, it was decided to assess the accuracy of each criterion by comparison with the results of a large number of beam-column tests.

Database of joint tests used to validate design criteria

The method chosen for assessing the validity of reinforcement anchorage design criteria was to assemble a large database of experimental results and consider the ability of design criteria to post-dict the structural performance of each experiment. The database of experimental results assembled for assessment purposes consisted of a large number of beam-column joint tests conducted internationally over the last four decades, resulting in the database and the method of assessment being similar to those used in previous studies conducted by Lin (1999) and by Fenwick and Megget (2003).

The criteria used to judge the suitability of a beam-column joint test for inclusion in the database were that the test consisted of a reinforced concrete interior beam-column joint subjected to a uniaxial cyclic loading history that included multiple inelastic cycles resulting in the joint performing in a manner consistent with the weak beam – strong column design ideal. In total 93 beam-column joint tests were included in the assembled database. A summary of each beam-column joint included in the database can be found in Table 2, while complete details of each joint are summarised elsewhere (Brooke 2011).

<u>Determination of structural design parameters</u>

For each beam-column joint included in the database, a large number of design parameters were recorded directly from the literature and further parameters were calculated from the available data. The majority of the analysis that was required before including a beam-column joint in the database was routine, and was based on standard New Zealand design methods. Examples of these routine calculations included determination of the predicted beam moment capacity and the joint shear strength. However a number of parameters were calculated using non-standard procedures, which are explained in the following sub-sections. A summary of the reported and calculated data for each beam-column joint is provided in Table 2 and the range of parameters covered in the database is summarised in Table 3.

Quantification of beam reinforcement asymmetry

All current rational anchorage design criteria recognise that the stress in beam compression reinforcement is affected if the areas of the beam top and bottom reinforcement groups differ. In design criteria this issue is accounted for by the inclusion of a parameter that is a function of β, being the ratio of the area of the compression reinforcement group to the area of the tension reinforcement group. It has been found (Brooke 2011) that there is inconsistency amongst international standards in the manner that β is defined. To simplify matters a second, closely related, variable ψ was introduced to represent the ratio of area of the smaller reinforcement group to the area of the larger reinforcement group.

For design purposes it is acceptable to define β or ψ as ratios of areas, but it is more precise to define these terms as ratios of the force resisted by the reinforcement groups. β was defined in the manner used by NZS 3101 (2006), i.e. as the area of the bar group containing the bar for which anchorage is being considered, divided by the area of the other bar group. Following the logic described by Restrepo-Posada (1993), slab reinforcement was assumed to not resist compression forces. Hence for bars in the top reinforcement group:

$$\beta_t = \frac{A_{st} f_{yt}}{A_{sb} f_{yb}} \tag{2}$$

where the subscripts t and b refer to top and bottom reinforcement respectively. For bars in the bottom reinforcement group:

$$\beta_b = \frac{A_{sb} f_{yb}}{A_{s,slab,n} f_{y,slab} + A_{st} f_{yt}} \tag{3}$$

where $A_{s,slab,n}$ is the area of slab reinforcement (if any) contributing to the nominal flexural strength of the beam according to NZS 3101 (2006), and $f_{y,slab}$ is the yield stress of the slab reinforcement. The ratio ψ was taken as the minimum of the values of β for the top and bottom reinforcement.

Performance assessment

Three forms of structural performance degradation can result from the occurrence of joint core anchorage failure (ACI Innovation Task Group 1 2001):

- Peak displacement strength degradation
- Small-displacement stiffness degradation
- Reduced energy dissipation, often described as hysteretic pinching.

Before analysing beam-column joints to assess how well the beam reinforcement was anchored in the joint core it was necessary to remove from consideration those joints which had their test performance limited by joint shear failure. This elimination process was based on the recorded force-displacement response of each joint and the reported test performance. A small number of tests were excluded that showed no signs of shear failure despite containing apparently only 5-15% of the joint reinforcement required by NZS 3101 (2006). Similarly, bond failure in beam-column joints was identified by examining the force-displacement response and noting the stage of testing at which excessive strength and/or small-displacement stiffness degradation occurred based on limits discussed below.

Unacceptable strength degradation was deemed to have occurred if the force resisted by the specimen at the peak of a cycle was less than 80% of the maximum force previously resisted in the same direction of loading (Park 1989). Small-displacement stiffness degradation was assessed against a criterion proposed by Fenwick & Megget (2003), who stated that unacceptable bond failure had occurred if the force resisted by a beam-column joint when half way to a target drift (i.e. lateral displacement divided by storey height) was less than 25% of the previous maximum resisted force in the same direction of loading. An alternative criterion (ACI Innovation Task Group 1 2001) was considered to be impractical to apply to large numbers of tests, particularly when the force-displacement responses were in many cases reproduced at small scales.

Definition of the bond failure displacement was made based on recognition that it is clearly more demanding to apply multiple cycles to the same displacement than to apply only a single cycle to each displacement, and it is reasonable to assume that a unit that failed during multiple cycles to a displacement slightly lower than a target displacement would have survived a single cycle to the target displacement (Fenwick and Dhakal 2007). To account for this behaviour a simple damage index was adapted from the work of Dhakal and Fenwick (2007). Thus the failure interstorey drift was calculated as:

$$\theta_{failure} = \left(\frac{\theta_{max} - \theta_{min}}{2} \right) \times 1.05^{(q-2)} \tag{4}$$

where θ_{max} and θ_{min} were taken as the peak positive and negative interstorey drifts applied to the unit during the last two half cycles before the half cycle in which failure occurred. The cumulative effect of multiple cycles was accounted for by the factor 1.05 and variable q, which was the number of half cycles previously completed to the same displacement as the half cycle before failure. While the method used to calculate $\theta_{failure}$ is not a particularly sophisticated damage index, the failure drifts calculated for each beam-column joint were considered representative of performance and hence the method proposed was deemed to be adequate.

Anchorage performance requirements for limit states

Structural performance is typically judged by New Zealand Standards (NZS 1170.5 2004) at three different limit states, termed the serviceability limit state (SLS), ultimate limit state (ULS), and maximum considered event (MCE). Consideration of anchorage performance in relation to these limit states lead to the following conclusions about performance requirements:

No anchorage deterioration should occur at the SLS. However, no damage was noted during the considered tests at drift levels commensurate with the SLS, i.e. up to approximately 1.0%. Thus no further specific consideration was given to this limit state.

Anchorage deterioration at the ULS should not exceed the limits discussed previously. According to New Zealand Standards (NZS 1170.5 2004), the design interstorey drift ratio should not exceed 1/0.7 × 2.5 = 3.57% (Fenwick and Megget 2003) at the ULS. Achievement of this drift level without excessive performance degradation was used as the basis for the assessments described here.

The performance objective for the MCE is collapse avoidance. The performance of beam longitudinal reinforcement anchorages need not be considered explicitly at this limit state as anchorage failure is unlikely to cause a structure to collapse (Fenwick and Megget 2003).

Performance of beam-column joints in the database

Table 4 summarises the performance of the assessed beam-column joints. It was determined that shear failure occurred in 29 of the 93 joints included in the assembled database, with 37 of the remaining joints having their performance limited by bond failure, which occurred at calculated drifts of 1.29% to 6.71%. 17 of these bond failures occurred when the drift was significantly less than the required ULS level of 3.57%, and 16 cases occurred when the drift was significantly greater than the required ULS level. This left four joints that failed when the drift was approximately equal to (i.e. within 0.1% of) the required ULS drift level. Identification of these joints with "marginal" performance provided additional information when working with the database of results, and also recognised the rather precise threshold (3.57%) that was used to determine performance when the measures underpinning the ascertained performance (strength and stiffness degradation) were rather imprecise and subjective.

Relating design criteria to experimental data

Values calculated using design criterion based on specimen details were related to the performance of that specimen. This comparison was achieved by using the design criterion being considered to calculate a demand to capacity (D/C) ratio for the joint core beam longitudinal reinforcement anchorages of each test specimen and then determining whether the performance indicated by the D/C ratio correlated with the performance that was assessed to have occurred during testing.

The D/C ratio used for most purposes was the ratio of required bond strength to available bond strength. The required and available bond strengths ($u_{b,req}$ and $u_{b,avail}$ respectively) were calculated for the top and bottom beam reinforcement of each beam-column joint as:

$$u_{b,avail} = \alpha_p u_b \tag{5}$$

$$u_{b,req} = \left(f_{s,max} + \kappa\alpha_o f_y\right)\frac{d_b}{4h_c} \tag{6}$$

with values for α_p, u_b, and κ taken from Table 1. The ACI 318 criterion could not be assessed using this D/C ratio because the criterion is not derived from the mechanics of force equilibrium. The best alternative D/C ratio was determined to be the ratio of required to available column depth ($h_{c,req}/h_{c,avail}$).

A weakness of the method used to investigate anchorage performance was that it was not possible to precisely identify whether bond failure affected the top, bottom, or both top and bottom reinforcing bars during testing. It was therefore necessary to determine for each beam-column joint whether the top or bottom reinforcing bar anchorages were more highly stressed and hence more likely to be the cause of failure. This determination was achieved by calculating the D/C ratios for the top and bottom reinforcing bars and then assuming that the reinforcing bars with the greater ratio would be the initial cause of bond failure during testing. The calculations for the different design criterion generally gave the same answer as to which bar was critical.

<u>Post-diction of anchorage performance using existing design criteria</u>

Figure 3 to Figure 6 show data points for each beam-column joint calculated using the NZS 3101, AIJ, EC 8, and ACI 318 design criteria respectively. It is immediately apparent that the data are widely scattered, with a significant range of D/C ratios that could result in any of satisfactory, marginal, or poor performance. For example, Figure 3 shows that either satisfactory or poor performance could occur in joints with anchorage D/C ratios between 0.81 and 1.24 calculated using NZS 3101. This observation is not unexpected for experimental data of this type, but makes determination of the adequacy of each design criterion more difficult. The position of the regression lines in Figure 3 to Figure 6 allows judgements to be made about the conservatism of the criteria, and the slope of the line allows assessment of the hypothesis of other researchers that high strength reinforcement requires disproportionately long anchorages in order to achieve the same level of performance as low strength reinforcement.

The regression lines in Figure 3, Figure 4, and Figure 5 were determined by plotting a straight line and requiring that no more than 5% of the data points below the line represented a beam-column joint that experienced premature bond failure or that had marginal performance. The slope and intercept of the regression line were determined by minimisation of an error function, with the value of the error function dependent on whether a joint performed satisfactorily or not. If a data point represented a joint that performed satisfactorily, then the error was taken as zero if the data point fell below the regression line, and was taken as $u_{b,req}/u_{b,avail} - r(f_y)$ if the data point was above the regression line, where $r(f_y)$ was the value of the regression line function. For data points representing joints that had marginal performance or had experienced premature bond failure the error function was reversed, i.e. it was taken as zero if the data point lay above the regression line. Data defining each of the regression lines is presented in Table 5 and full details of the process are presented elsewhere (Brooke 2011).

It was not possible to calculate a regression line for the data obtained using the ACI 318 criterion due to the plotted data points for joints with satisfactory, marginal, and unsatisfactory performance being completely intermingled. The ACI 318 criterion is also extremely non-conservative, with failure occurring in joints with D/C ratios as low as 0.62.

Several observations regarding the design criteria can be drawn from the data in Table 5 and from Figure 3 to Figure 5:

- None of the existing design criteria are particularly well defined.

- The NZS 3101 design criterion is non-conservative. According to this criteria a joint would require a D/C ratio less than approximately 0.88 to have a low probability of premature bond failure.
- The other rational design criteria are conservative by approximately 10% because satisfactory anchorage performance would be expected even if the D/C ratio was approximately 1.10.

It was concluded that a revised design criterion should be developed by analysis of the experimental data, based on the observation that none of the existing criterion correlated well to the experimental data.

Parametric assessment of anchorage requirements in interior joints

A revised design criterion was developed having the same equilibrium basis as the NZS 3101, AIJ, and EC 8 design criteria and shown in Figure 1. An underlying assumption when developing the revised criterion was that the complex real distribution of bond stresses that exist in a beam-column joint can be adequately modelled as an average bond stress acting along the length of the reinforcing bar passing through the joint (i.e. a length equal to the total column depth). Preliminary investigation suggested that more complex alternatives would not have enhanced the accuracy of the design criterion.

The revised design criterion was developed using a parametric assessment method because the number of factors that influence the performance of reinforcement anchorages at interior beam-column joints is sufficiently large that other assessment methods are impractical. The principal variables considered were:

- Concrete strength, which is related to the basic available bond strength
- Column axial load, which is known to affect the available bond strength
- Beam reinforcement asymmetry, which is known to affect the compression reinforcement stress and hence the force to be anchored.

Basic available bond strength

It is generally recognised that the maximum bond stress that can be developed between steel reinforcement and concrete in a beam-column joint core is related to the tensile strength of concrete, and hence proportional to the compressive strength of concrete raised to an exponent with a value between zero and one. Thus if the effects of column axial load, fresh concrete depth, and loading type are included by multipliers (respectively α_p, α_t, and α_f,), the bond strength available in a joint core can be expressed as:

$$u_{b,avail} = \alpha_f \alpha_t \alpha_p u_b = \alpha_f \alpha_t \alpha_p \eta \left(f_c'\right)^n \tag{7}$$

where $0 < n < 1$ and η is a constant. In order to relate equation 7 to experimental data, it is also necessary to know the bond strength required in each test unit (using equation 6). Considering equations 6 and 7, only two of the factors in the equations are known, i.e. the concrete compressive strength and the maximum reinforcement tension stress, but the relationship for the basic bond strength as a function of concrete compressive strength can be determined subject to a number of assumptions:

- The variable α_p is dependent on the column axial load and equal to 1.0 for insignificant axial loads (i.e. when $N/A_g f_c' \leq 0.05$.
- The variable α_f is equal to 1.0 when a joint is subjected to uni-directional loading.
- The variable α_t has values specified by NZS 3101, i.e. 1.0 if the depth of fresh concrete cast under a bar is less than 300 mm or 0.85 otherwise (Cheung 1991).
- The compression reinforcement stress is controlled by the variable $\kappa\alpha_o$,that is dependent on β and is equal to $0.7f_y$ when $\beta = 1.0$ (Cheung 1991).

To take advantage of these assumptions a subset of joints with equal top and bottom beam reinforcement ($\beta = 1.0$) and no significant column axial compression load (defined as $N/A_g f_c' \leq 0.05$) was selected from the experimental database. The number of beam-column joints in this subset was nineteen, including five joints in which premature bond had failure occurred. None of the nineteen joints were amongst those determined to have marginal performance. Figure 7 shows $\log(u_{b,req})/\alpha_t$ plotted against $\log\left(f_c^{'}\right)$ for the 19 beam-column joints with no column axial load and β = 1.0. The use of a log-log scale in Figure 7 means that the slope and intercept of a straight line drawn on the graph can be used to determine the values of η and n.

A fitting method similar to that discussed previously was used to determine that the bond strength that can be reliably depended upon is:

$$u_{b,avail}/\alpha_t = 1.25\left(f_c^{'}\right)^{0.5} \qquad (8)$$

noting that some rounding has been applied to the values of n and η obtained by minimising the error function in order to give a more practical equation and to avoid implying undue accuracy. Equation 8 was used as the basis of further development of a design criterion described in the remainder of this section.

Effect of column axial load on available bond strength

Having defined an equation for the basic available bond strength in the absence of column axial load, a relationship between available bond strength and column axial load was next defined by selecting an expanded subset of experimental results containing beam-column joints that have equal top and bottom reinforcing group areas and any value of column axial load. This enlarged subset included 37 joints, of which one joint had marginal performance and eight joints had premature anchorage failures.

By combining and rearranging the equations used to calculate available bond strength (equation 7), the derived relationship between concrete strength and bond strength (equation

8), and the required bond strength (equation 6), an equation was developed that allowed the bond strength modifier α_p to be calculated for each beam-column joint:

$$\alpha_p = \frac{d_b}{1.25\alpha_t h_c \left(f_c^{'}\right)^{0.5}} \left(f_{s,max} + 0.7 f_y\right) \quad (9)$$

The value of α_p calculated according to the above equation is the value required for the available bond strength to exactly equal the required bond strength. Obviously the required and available bond strengths are not equal in most beam-column joints, as in joints where anchorage failure occurred the required bond strength exceeded the available bond strength, and in beam-column joints that performed satisfactorily the available bond strength exceeded the required bond strength. Thus the calculated α_p values are fictitious, but can be used as a guide to indicate how the usable bond strength changes if axial load is applied to a column.

Figure 8 shows the calculated values of α_p plotted against column axial load for the 37 beam-column joints being considered. It is evident that the data in Figure 8 is poorly constrained due to the limited number of tests conducted in which column axial loads were applied. However, the data does support the common view that increasing axial load increases the usable bond strength.

Existing design criteria assume that bond strength increases linearly with column axial load. However, tests on isolated bars embedded in concrete suggest that there is an upper limit to the bond strength enhancement that can be achieved by increasing the transverse compression stress acting on an anchorage (fib Task group Bond Models 2000). The maximum enhancement corresponds to a finite compression stress, beyond which further increase of the compression stress does not result in a corresponding increase in bond strength. A proposed relationship between column axial load ratio and α_p based on this knowledge is plotted as a dotted line on Figure 8 and is defined by the following equation:

$$\alpha_p = 0.9 + 2\frac{N}{A_g f_c^{'}} \qquad (10)$$

with limits that $1.0 \leq \alpha_p \leq 1.2$.

<u>Effect of asymmetric beam reinforcement on anchorage performance</u>

The final step required to fully define a revised anchorage design criterion is a relationship between β and beam reinforcement compression stress. In order to isolate beam reinforcement asymmetry a third subset including joints with no column axial load and having any value of β was used. The subset selected consisted of 31 beam-column joints, including eight joints in which premature bond failure occurred and one joint that had marginal performance.

Assessment of the level of compression reinforcement stress that should be assumed to occur at the face of a beam-column joint required the variable $\kappa\alpha_o$ to be isolated. By combining and rearranging the equations used to calculate available bond strength (equation 7), the derived relationship between concrete strength and bond strength (equation 8), and the required bond strength (equation 6), and taking $\alpha_p = 1.0$ as $N/A_g f_c^{'} > 0.05$ for all the joints being considered, the following relationship was derived:

$$\kappa\alpha_o \leq \frac{5\alpha_t h_c \left(f_c^{'}\right)^{0.5}}{f_y d_b} - \frac{f_{s,max}}{f_y} \qquad (11)$$

While equation 11 appears to be fairly abstract, it can be physically interpreted as stating that in order to ensure satisfactory performance, the bond stress required to anchor the reinforcement compression force should be less than or equal to the available bond stress minus the bond stress required to anchor the reinforcement tension force.

Before equation 11 was used to analyse the database of beam-column joint test results, the expected form of the function of β representing $\kappa\alpha_o$ was considered by examining the equilibrium of flexural tension and compression forces for beam section analyses at the faces of a beam-column joint (Brooke 2011). This consideration lead to the conclusion that a rational expression for $\kappa\alpha_o$ applicable to the anchorage of top or bottom reinforcement is given by equation 12:

$$\kappa\alpha_o = \frac{0.7}{\beta} \leq 1.0 \tag{12}$$

The restriction that $\kappa\alpha_o \leq 1.0$ has the effect of limiting the compression reinforcement stress to a maximum value of f_y. This restriction reflects previous New Zealand research and practice that suggests that strain hardening of reinforcement in compression is not likely to occur (Paulay and Priestley 1992).

Equation 12 was compared with values of $\kappa\alpha_o$ calculated using equation 11 for each of the 31 beam-column joints that had insignificant column axial compression load. The calculated values of $\kappa\alpha_o$ and equation 12 are plotted in Figure 9. Values of β for the joints considered range from 0.40 to 2.05.

Interpretation of Figure 9 is best understood by recollection of the previous interpretation of equation 11. The plotted values of $\kappa\alpha_o$ equate to the magnitude of the compression reinforcement stress needed for the required bond strength to equal the available bond strength. Thus for data points where $\kappa\alpha_o$ >1.2, bond failure would not be expected to occur unless compression reinforcement stresses greater than $1.2f_y$ occurred, which is unlikely. Conversely $\kappa\alpha_o = 0.4$ indicates that a compression reinforcement stress of $0.4f_y$ would overstress the anchorage.

The data shown in Figure 9 supports previously reported trends:

- If beams have equal areas of top and bottom reinforcement, bond failure is unlikely to occur unless the compression reinforcement stress required to overstress the anchorage is less than approximately $0.7f_y$.
- When the area of compression reinforcement is smaller than the area of tension reinforcement ($\beta < 1.0$), several bond failures occurred when the compression reinforcement stress required to overstress the anchorage was greater than $0.7f_y$, supporting the postulation that greater compression reinforcement stresses can occur when $\beta < 1.0$. However, and significantly, bond failure was not observed to occur when the compression reinforcement stress required to overstress the anchorage exceeded the reinforcement yield stress.
- Several reinforcement anchorages with $\beta > 1.0$ performed well even though the compression reinforcement stress required to overstress the anchorage was less than 0.5fy. This observation supports the postulation that compression reinforcement stresses are reduced to low levels if β is greater than 1.0.

The proposed relationship between β and compression reinforcement stress (equation 12) provides an appropriate upper bound to the stress that occurred during beam-column joint tests.

Proposed design equation

Figure 10 shows the ratio of required to available bond strength for each of the beam-column joints calculated using the relationships developed in the previous sections. The developed relationships result in a good prediction of experimental performance, as is shown by the regression line also plotted in Figure 10. The regression line plotted in Figure 10 is defined by a value of 1.001 when the beam reinforcement yield stress is 300 MPa and a slope of -85.2x10-6/MPa. The total error sum for the regression line was calculated as 0.137. Comparison of these values with those shown in Table 5 leads to the following conclusions:

- The proposed design criterion predicts the experimental performance of beam-column joints more accurately than existing criteria.
- The scatter of experimental data is lower for the proposed criterion than for other criteria.

Having shown that the proposed equation is able to adequately predict the behaviour of experimental results, it is necessary to present the equation in a form suitable for design use. The following design requirement is appropriate:

$$\frac{d_b}{h_c} \leq 5 \frac{\alpha_p \alpha_f \alpha_t}{\alpha_s} \frac{\sqrt{f_c^{'}}}{\alpha_o f_y} \qquad (13)$$

where $\alpha_t = 0.85$ except if the depth of concrete cast under a bar is less than 300 mm in which case $\alpha_t = 1.0$, α_o is the reinforcement overstrength factor, and α_p is given by equation 10. The coefficient α_f accounts for the effect of bi-directional loading on anchorage performance. Values for α_f are taken to be those suggested by NZS 3101, i.e. 1.0 unless a joint is subjected to bi-directional loading in which case $\alpha_f = 0.85$.

Following current New Zealand terminology, the coefficient α_s defines the level of compression reinforcement stress to be resisted by an anchorage. The equation for α_s used here has not previously been derived. However, working from the total reinforcement force to be anchored, α_s must be such that:

$$\alpha_s = 1 + \frac{0.7}{\alpha_o \beta} \leq 1 + \frac{1}{\alpha_o} \qquad (14)$$

Conclusions

Comparison and assessment of the accuracy of existing international anchorage design criteria for interior beam-column joints has shown that significant differences exist between these criteria, and that existing criteria were do not realistically reflect experimental performance of beam-column joints. A revised criterion was developed based on the experimental performance of joint core anchorages. The revised criterion has been shown to ensure that beam-column joints achieve a required level of performance with an appropriate degree of reliability, i.e. a 95% likelihood of bond failure not occurring. The revised criterion is empirical and limited to the data available from tests. While the data considered covers a wide range of beam-column joints, care should be taken before extrapolating the resulting equation to joints with parameters outside of the range considered.

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Table 1: Comparison of key aspects of international design criteria

Design criteria	Compression reinforcement stress, $\kappa\alpha_o f_y$ for bars that are part of the:		Basic average bond stress, u_b	Bond stress multiplier from column axial stress, α_p
	larger group	smaller group		
AIJ	$\psi\alpha_o f_y$	$\alpha_o f_y$	$0.69\left(f_c'\right)^{2/3}$	$1.0+\frac{P}{A_g f_c'}$
NZS 3101:1995	$0.55\alpha_o fy$	$(1.55-\psi)\alpha_o f_y$, ($\psi \geq 0.75$)	$\alpha_f \alpha_t 1.5\sqrt{f_c'}$	$0.95+\frac{0.5P}{A_g f_c'} \geq 1.0$
EC 8	$0.75\psi\alpha_o f_y$	$0.75\alpha_o f_y$	$0.56\left(f_c'\right)^{2/3}$	$1.0+\frac{0.8P}{A_g f_c'}$

Table 2: Summary information for beam-column joint test units

Research programme	Unit	f_y (MPa)	d_b (mm)	β	h_c (mm)	f_c' (MPa)	$\frac{N}{A_g f_c'}$	Failure mode	Bond failure drift
Amso (2005)	1	564	16.0	1.00	360	29.3	0.00	Bond	2.0%
	2	584	16.0	1.00	360	40.4	0.00	Bond	4.4%
	3	585	16.0	1.00	360	40.9	0.00	Bond	3.0%
	4	595	20.0	1.00	360	53.0	0.00	Bond	4.0%
Beckingsale (1980)	B11	298	19.1	0.50	457	35.9	0.04	None	-
	B12	298	19.1	1.00	457	34.6	0.04	None	-
	B13	298	19.1	1.00	457	31.4	0.26	None	-
Brooke (2011)	1B	552	25.0	1.00	360	31.2	0.00	None	-
	2B	552	25.0	1.00	360	40.6	0.00	None	-
	3B	543	25.0	1.00	360	44.8	0.00	Bond	4.4%
	4B	543	25.0	1.00	360	42.8	0.00	Bond	4.4%
	M.PC	552	25.0	1.00	400	38.0	0.00	Bond	3.0%
Central Labs (Lawrance et al. 1991; 1993; Stevenson and Beattie 1988; 1989)	L91	336	20.0	1.00	500	28.9	0.00	Bond	3.0%
	L93	466	20.0	2.01	300	83.3	0.00	Bond	6.7%
	S88	315	28.0	1.01	500	39.6	0.04	None	-
	S89	303	28.0	1.00	650	47.5	0.01	None	-
Cheung (1991)	1D-1	283	24.0	0.55	550	38.0	0.00	None	-
Dai (Park and Dai 1988)	1	294	16.0	0.40	305	45.9	0.00	None	-
	2	300	20.0	2.05	305	36.0	0.00	Bond	4.4%
	3	294	16.0	0.40	305	36.2	0.00	None	-
	4	300	20.0	2.05	305	40.1	0.00	Bond	3.6%
Durrani (1982; 1985)	S1	336	22.2	0.56	362	41.6	0.06	Shear	-
	S2	336	22.2	0.56	362	30.8	0.08	Shear	-
	S3	331	22.2	0.48	362	28.3	0.06	Bond	3.6%
	X1	331	22.2	0.77	362	34.3	0.05	Shear	-
	X2	331	22.2	0.77	362	33.6	0.06	Shear	-
	X3	331	22.2	0.77	362	31.0	0.05	Bond	2.7%
Englekirk(1998a; 1998b; 2003; Pourzanjani and Englekirk 2000)	12-1	460	34.9	1.00	610	88.3	0.03	Shear	-
	12-2	460	34.9	1.00	610	82.0	0.07	Shear	-
	12-3	460	34.9	1.00	610	68.3	0.04	None	-
	6-1	460	34.9	1.00	610	49.6	0.06	Shear	-
Joh (1991a; 1991b; Kurose 1987)	B1	371	12.7	1.00	300	21.3	0.16	Shear	-
	B2	371	12.7	1.00	300	20.8	0.17	Shear	-
	B4	371	12.7	0.67	300	21.7	0.16	Shear	-
	B5	371	12.7	1.00	300	23.1	0.15	Shear	-

Table 2 (continued): Summary information for database of beam-column test units

Research programme	Unit	f_y (MPa)	d_b (mm)	β	h_c (mm)	f'_c (MPa)	$\frac{N}{A_g f'_c}$	Failure mode	Bond failure drift
Joh	HH	404	12.7	1.00	300	25.6	0.15	Bond	4.1%
	HL	404	12.7	1.00	300	27.4	0.15	Bond	4.1%
	LH	404	12.7	1.00	300	26.9	0.15	Bond	4.1%
	MH	404	12.7	1.00	300	28.1	0.15	Bond	4.1%
Lin (1999)	U1	525	12.0	1.00	390	33.3	0.43	Shear	-
	U2	525	12.0	1.00	390	33.3	0.43	Bond	2.9%
	U3	525	12.0	1.00	390	37.0	0.10	Bond	3.6%
	U4	525	12.0	0.50	390	37.0	0.10	Bond	3.7%
	U8	525	12.0	1.00	390	33.2	0.10	Shear	-
Milburn (1982)	U1	315	16.0	1.00	305	41.3	0.10	None	-
Oka & Shiohara (1992)	J7	676	13.0	0.71	300	79.2	0.12	None	-
	J9	676	13.0	0.73	300	79.2	0.12	None	-
Priestley (1975)	P1	276	28.6	0.60	686	48.5	0.03	None	-
Restrepo (1993)	U5	285	24.0	0.87	450	27.0	0.00	Bond	2.4%
Restrepo	U6	285	24.0	1.00	450	44.0	0.00	None	-
Soleimani (1978)	BC3	492	15.9	0.52	432	31.1	0.36	Bond	4.6%
Teraoka (2005; 1997)	HJ1	382	19.1	1.00	400	54.2	0.20	None	-
	HJ2	624	15.9	1.00	400	54.2	0.20	Bond	4.4%
	HJ3	858	19.1	1.00	400	54.2	0.20	Bond	3.0%
	HJ4	382	19.1	1.00	400	54.2	0.20	Shear	-
	HJ5	645	19.1	1.00	400	54.2	0.20	Bond	4.2%
	HJ6	858	19.1	1.00	400	54.2	0.20	Bond	3.2%
	HJ7	422	22.2	1.00	400	92.6	0.20	None	-
	HJ8	599	22.2	1.00	400	92.6	0.20	None	-
	HJ9	858	19.1	1.00	400	92.6	0.20	None	-
	HJ10	611	15.9	1.00	400	88.7	0.20	None	-
	HJ11	441	22.2	1.00	400	88.7	0.20	Shear	-
Teraoka	HJ12	604	22.2	1.00	400	88.7	0.20	Shear	-
	HJ13	625	19.1	1.00	400	116.9	0.20	None	-
	HJ14	604	22.2	1.00	400	116.9	0.20	Shear	-
	HJ15	776	22.2	1.00	450	138.2	0.20	Shear	-
	HJ16	776	22.2	1.00	450	138.2	0.20	Shear	-
University of Tokyo (Kitayama et al. 1992; 1991; Kurose 1987; Lee et al. 1992)	C1	320	9.5	0.50	300	25.6	0.08	None	-
	C2	320	9.5	0.50	300	25.6	0.08	None	-
	C3	320	9.5	0.50	300	25.6	0.08	None	-

Table 2 (continued): Summary information for database of beam-column test units

Research programme	Unit	f_y (MPa)	d_b (mm)	β	h_c (mm)	$f_c^{'}$ (MPa)	$\frac{N}{A_g f_c^{'}}$	Failure mode	Bond failure drift
University of Tokyo (Kitayama et al. 1992; 1991; Kurose 1987; Lee et al. 1992)	I1	799	15.9	1.00	300	98.8	0.04	Shear	-
	I3	361	15.9	0.67	300	41.4	0.03	Shear	-
	I4	370	15.9	0.67	300	39.7	0.03	Shear	-
	I5	769	12.7	0.50	300	85.4	0.02	Shear	-
	I6	772	19.1	0.67	300	85.4	0.02	Shear	-
	J1	401	12.7	0.50	300	25.7	0.08	Shear	-
	J2	401	12.7	0.50	300	24.0	0.08	Shear	-
	J3	401	12.7	0.50	300	24.0	0.08	Shear	-
	J4	401	12.7	0.50	300	25.7	0.23	Shear	-
	J5	401	12.7	0.50	300	28.7	0.07	Shear	-
	J6	401	12.7	0.75	300	28.7	0.07	Bond	2.3%
	S1	344	12.7	0.75	300	27.8	0.21	Bond	2.3%
	S2	344	12.7	0.75	300	27.8	0.07	None	-
	S3	417	9.5	0.71	300	27.8	0.21	Bond	2.3%
	S4	371	15.9	0.67	300	25.1	0.23	Bond	2.2%
	S6	344	12.7	0.75	300	25.1	0.23	Bond	2.2%
Xin (1992)	U1	453	12.0	1.00	300	30.9	0.00	None	-
	U2	445	16.0	0.50	300	40.8	0.00	Bond	1.3%
	U3	445	16.0	1.00	300	42.5	0.00	Bond	4.4%
	U4	492	20.0	1.73	300	47.2	0.00	Bond	3.4%
	U5	492	20.0	1.00	300	60.7	0.00	Bond	3.4%
	U6	492	20.0	1.84	300	59.3	0.00	Bond	4.0%
Young (1998)	U1	519	16.0	1.00	300	49.2	0.00	Bond	4.4%

Table 3: Range of structural parameters covered by database

Parameter	Range
Beam reinforcement yield stress	265 – 858 MPa
Beam reinforcement diameter	9.5 – 35 mm
Reinforcement ratio β	0.4 – 2.5
Concrete strength	20.8 – 138 MPa
Beam depth	300 – 900 mm
Column depth	300 – 1000 mm
Anchorage length h_c/d_b	14.5 – 37.5
Column axial load ratio, $\frac{N}{A_g f_c'}$	0 – 0.43

Table 4: Summary of beam-column joint database

	Number of beam-column joints
Total included in database	93
Joint shear failure	29
Available for bond strength analysis	64
Premature bond failure	17
Bond failure when drift = 3.57±0.1%	4
Satisfactory performance	43

Table 5: Regression data for current anchorage design criteria

Design criterion	$u_{b,req}/u_{b,avail}$ at fy = 300 MPa	Slope (×10-6/MPa)	ΣErrTOT
NZS 3101	0.874	-85.7	0.226
AIJ	1.081	217.0	0.273
EC 8	1.190	176.7	0.325

Figure 1
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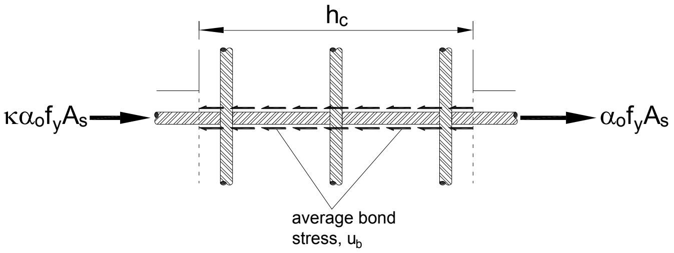

Figure 1: Free body diagram of a reinforcing bar anchored in a beam-column joint core

Figure 2
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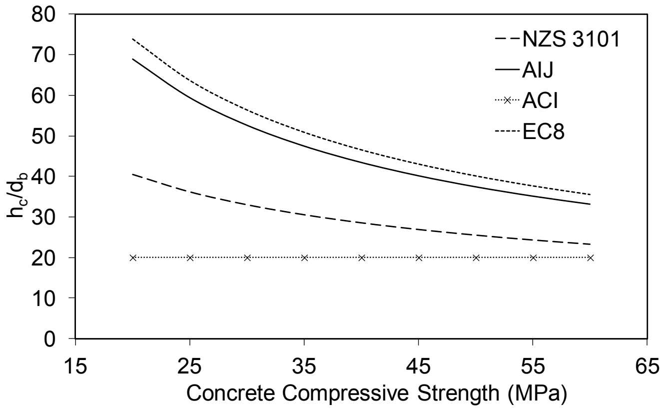

Figure 2: Comparison of international design requirements for "reference" joint

Figure 3
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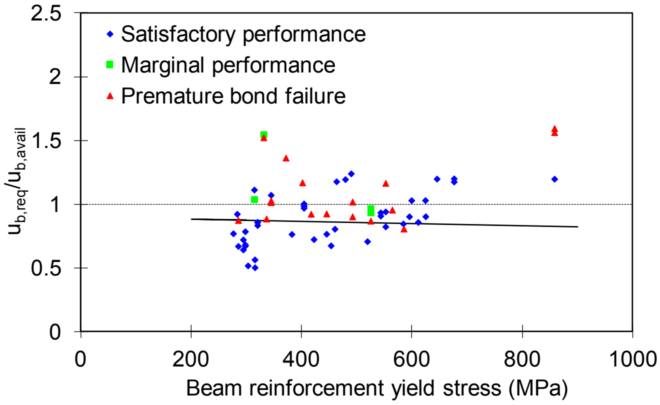

Figure 3: Prediction of beam reinforcement anchorage performance using the NZS 3101 design criterion

Figure 4
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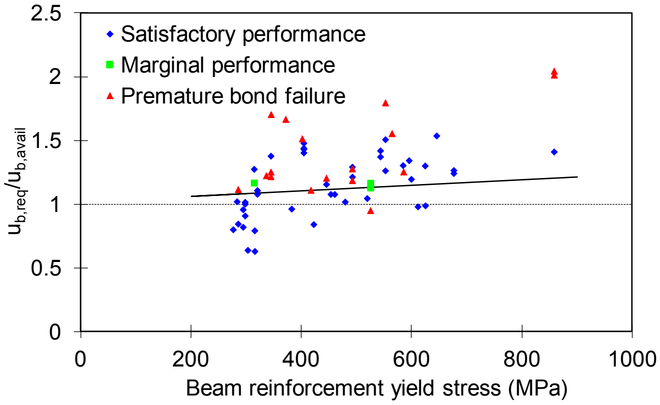

Figure 4: Prediction of beam reinforcement anchorage performance using the AIJ design criterion

Figure 5
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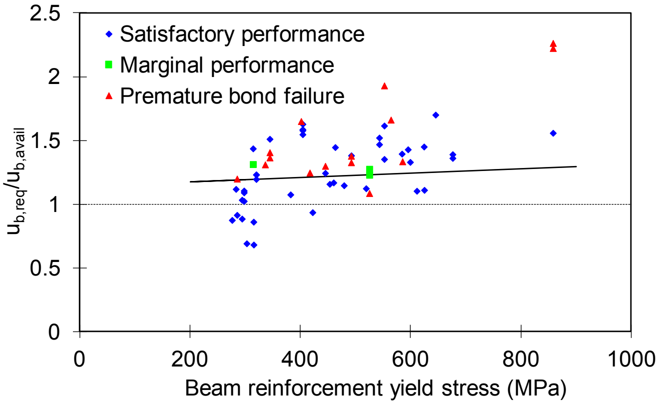

Figure 5: Prediction of beam reinforcement anchorage performance using the EC 8 design criterion

Figure 6
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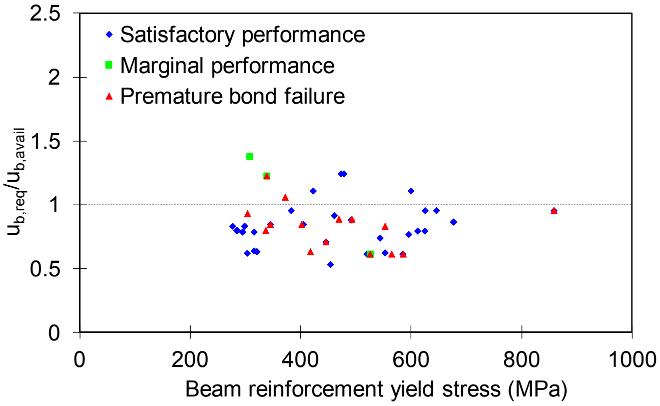

Figure 6: Prediction of beam reinforcement anchorage performance using the ACI 318 criterion

Figure 7
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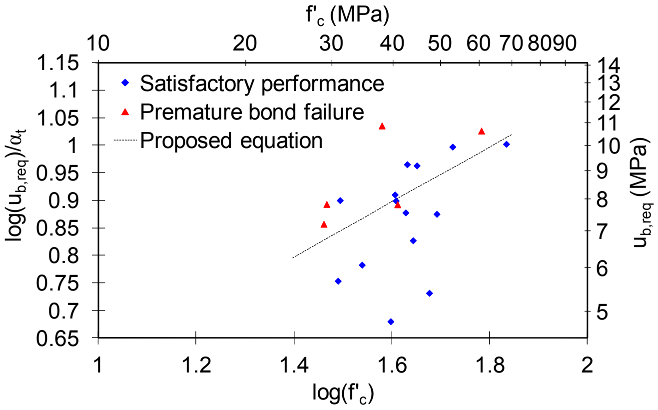

Figure 7: Comparison of bond strength and concrete strength for beam-column joints with equal top and bottom reinforcement and no column axial load

Figure 8

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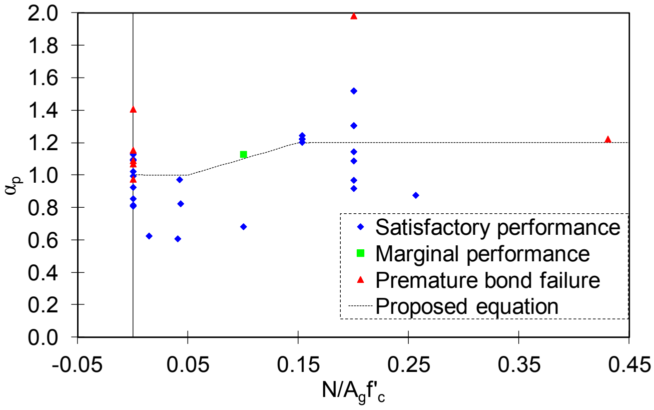

Figure 8: Comparison of bond strength and column axial load for beam-column joints with equal top and bottom beam reinforcement

Figure 9
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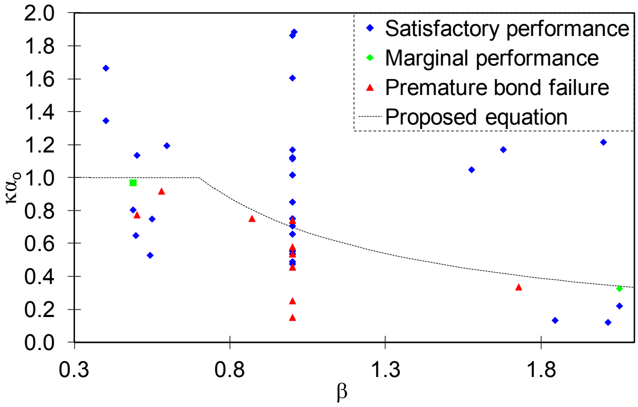

Figure 9: Comparison of reinforcement ratio and bond strength for beam-column joints with no column axial load

Figure 10
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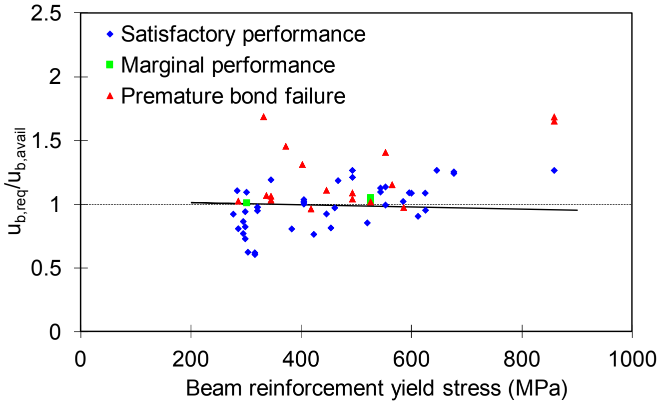

Figure 10: Prediction of beam reinforcement anchorage performance using proposed equation

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Figure 5: Prediction of beam reinforcement anchorage performance using the EC 8 design criterion

Figure 6: Prediction of beam reinforcement anchorage performance using the ACI 318 criterion

Figure 7: Comparison of bond strength and concrete strength for beam-column joints with equal top and bottom reinforcement and no column axial load

Figure 8: Comparison of bond strength and column axial load for beam-column joints with equal top and bottom beam reinforcement

Figure 9: Comparison of reinforcement ratio and bond strength for beam-column joints with no column axial load

Figure 10: Prediction of beam reinforcement anchorage performance using proposed equation