EVALUATION OF THE SHEAR STRENGTH OF BEAM-COLUMN JOINTS OF REINFORCED CONCRETE FRAMES SUBJECTED TO EARTHQUAKE LOADING

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SUMMARY

Since 1967, when Hanson and Conner [1967] carried out the first seismic loading test on a beam-column joint of a reinforced concrete moment resisting frame, the importance of the design of beam-column joints has been recognised in the seismic design recommendations in different building standards. Recent earthquakes have strengthened the need for properly reinforcing beam-column joints to avoid partial or total structural collapse in large events and to avoid irreparable damage in moderate events. It is interesting to note that 30 years after the first test, and after a great deal of experimental work, recommendations for the design of beam-column joints in different design standards still have many discrepancies. This difference can be partly attributed to the difficulties in identifying the main parameters that affect the behaviour of joints.

A behavioural model is used in this paper to identify the main variables that influence the behaviour and strength of interior beam-column joints of reinforced concrete frames designed for earthquake resistance. The paper also examines a database and proposes equations for the design of the horizontal joint reinforcement.

INTRODUCTION

Shear forces in interior beam-column joints of typical moment resisting frames can be of the order of magnitude four to six times larger than the shear forces of the framing columns. This level of forces invariable leads to rather large shear stresses. Joints are part of the vertical load carrying system and are known to have very poor energy dissipation characteristics. Consequently, concentration of plasticity in joints is considered undesirable.

Building design standards in different locations provide recommendations for the seismic design of joints at the Ultimate Limit State. The aim of the design recommendations is to ensure satisfactory performance of joints during a strong earthquake. The main problem is that the behaviour of beam-column joints subjected to reversed cyclic loading is not understood as clearly as the behaviour of other reinforced concrete members, and certainly, an accurate prediction of the joint shear strength is still difficult. This problem, combined with the lack of proper performance design criteria, has led to very different specifications in design standards worldwide. Design recommendation in different standards can be broadly classified in two main groups. One group [Eurocode 8, 1994; SNZ, 1995] bases the recommendations on the behavioural parallel angle steel truss and diagonal concrete strut transfer mechanisms proposed by Park and Paulay [1975]. The other group [ACI-318, 1995; AIJ, 1994] bases the design recommendations in a confinement criterion. The latter group tacitly recognises that transverse reinforcement does not enhance the joint shear strength.

The method proposed in this paper is aimed at improving the understanding of the force transfer mechanisms in beam-column joints. The paper discusses the mechanisms of force transfer and also the design of the horizontal reinforcement in interior beam-column joints. The same methodology be extended and used to study the force transfer mechanisms in exterior beam-column joints.

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DESCRIPTION OF THE ANALYTICAL MODEL

The analytical model described in this paper is based on the lower bound theorem of plasticity and uses strut-and-tie models [Schläich et al., 1994] to evaluate the internal force flow within a joint panel. Several variables, which are likely to affect the shear strength of the joints, were investigated to evaluate their relative importance [Lin et al., 1997; Lin, 1999]. The following assumptions were made:

1. The joint is a plane element,
2. The concrete in the joint panel is cracked,
3. All beams framing into the joint, including lateral beams if they exist, form plastic hinges at the joint faces,
4. The columns framing into the joint remain elastic,
5. The concrete compressive force resulting from flexure in the beam acts at the level of the longitudinal reinforcement closest to the extreme fibre in compression,
6. The column shear force is linearly distributed force throughout the effective depth of the column and is maximum at the column compressive end,
7. The beam shear force acts in the joint as a concentrated force,
8. Bond forces in the longitudinal beam bars passing through the joint region follow a bond stress law, and,
9. The centre of the joint is the critical region where failure occurs by crushing of the concrete after the entire horizontal joint reinforcement yields in tension.

The last assumption is based on observation made in laboratory tests where after several reversed load cycles and beam hinging at the joint faces, the entire horizontal joint reinforcement yields. Two exceptions can be found: (i) if the yield force resulting from the horizontal joint reinforcement exceeds the horizontal shear force, some hoops will remain elastic, and (ii) joint hoops placed next to the beam longitudinal reinforcement do no always yield in tension.

PARAMETRIC STUDY

The main variables that are believed to affect the stress distribution in the diagonal compression field of an interior beam-column joint with beams hinging at the joint faces are:

(a) The bond stress distribution along the longitudinal beam bars,

(b) The ratio A'_s / A_s, where A'_s and A_s are the areas of the beam bottom and top longitudinal reinforcement passing through the joint, respectively,

(c) The ratio V_{sh} / V_{jh}, where V_{sh} is the horizontal shear resistance provided by the horizontal joint reinforcement and V_{jh} is the horizontal joint shear force, and,

(d) The ratio $N^* / (A_g f'_c)$, where N^* is the axial compressive force acting on the column framing into the joint, A_g is the horizontal cross section area of the column and f'_c is the concrete cylinder compressive strength.

A parametric analysis was carried out to investigate the influence of variables (a) to (d). Column longitudinal bar forces were obtained from a moment-curvature analysis. Beam longitudinal bar tensile forces were calculated as the nominal yield force multiplied by an overstrength factor equal to 1.25. The effective joint area, needed to estimate the joint shear stress v_{jh}, was taken equal to the horizontal cross section area of the joint. The struts within the joint satisfied the boundary conditions and internal equilibrium. The diagonal compression field in the joint panel was modelled with five to seven main struts. The average uniaxial stress at the centre of the joint panel f_{cs}, was defined as the force carried by the central strut divided by the semi-distance between the struts at either side of the central strut (see Fig. 1). The stress f_{cs} does not represent the maximum uniaxial stress in the joint due to the discrete nature of the model. However, if the number of struts is kept relatively constant, this stress can be used to measure the relative importance of the different variables.

Figure 2 shows a five strut-and-tie model for two joints. These joints are identical except for the bond stress distribution of the beam longitudinal reinforcement. The bond stress distribution chosen for the joint shown in Fig. 2 (a) is representative of joints with relatively large h_c / d_b ratios, where h_c is the overall column depth and d_b is the nominal diameter of the bar. The bond stress distribution chosen for the joint shown in Fig. 2 (b) is more typical of joints with small h_c / d_b ratios. The bond force in both examples is the same. Despite the large variation in the bond stress distribution, little difference in the uniaxial compressive stress ratio f_{cs} / v_{jh} is observed. This finding contrasts with the sensitivity of the Park and Paulay model to the bond stress distribution [Paulay and Priesltey, 1992].

The role of the interior column reinforcement is also evident in the two cases shown in Fig. 2. Not only these contribute to resisting flexure in the column but also sustain the vertical component of the diagonal compression field in the joint panel. The interior bars are subjected to larger tensile stresses when better beam bar anchorage conditions exist, compare the examples shown in Fig. 2 (a) and (b). It can be shown that if little vertical reinforcement in the way of interior column longitudinal bars is provided, the bars will yield prematurely and only small bond forces can develop along the beam bars, except where they are clamped by the column compressive stress block. The effect of the column interior bars on the stress ratio f_{cs} / v_{jh} is small, except if no or little horizontal joint reinforcement is provided [Lin, 1999].

The analytical model indicates that the ratio A'_s / A_s has a very small influence in the stress ratio f_{cs}/v_{jh} [Lin, 1999].

Figure 3 depicts two strut-and-tie models in which the ratio N^* / ($A_g f_c^{'}$) was varied. The parametric analysis showed that the column axial load influences the stress ratio f_{cs} / v_{jh}. At low axial load levels, an increase in axial compression reduces the stress ratio f_{cs} / v_{jh}, but this ratio begins to increase with an increase in axial compression when N^* / ($A_g f_c^{'}$) > 0.3. This trend is particularly accentuated if little horizontal joint reinforcement is provided. This is because a corner-to-corner diagonal strut, that can very easily be overloaded, carries most of the joint shear force.

The analysis also showed that the ratio V_{sh} / V_{jh} also influences the stress ratio f_{cs} / v_{jh}. When the ratio V_{sh} / V_{jh} is small the internal forces flow mainly through a corner-to-corner diagonal strut. A more evenly internal force flow is observed when the ratio V_{sh} / V_{jh} is moderate or large.

The combined effect of ratios N^* / ($A_g f_c^{'}$) and V_{sh} / V_{jh} is plotted in Fig. 4. The dots in the graph correspond to a strut-and-tie solution. Five strus were used to analyse the joints with V_{sh} / V_{jh} = 0 whereas the joints while the remaining joints were analised with five, or seven struts. It can be inferred from this figure that the stress f_{cs} is approximately equal to 0.3 $f_c^{'}$ in a joint with V_{sh} / V_{jh} = 1 and no column axial load, when $v_{jh} = 0.1f_c^{'}$. Taking another example, the stress ratio f_{cs} / $f_c^{'}$ in an unreinforced beam-column joint (V_{sh} / V_{jh} = 0) with N^* / ($A_g f_c^{'}$) = 0.4 is approximately 2.8 times the stress ratio f_{cs} / $f_c^{'}$ of a joint with V_{sh} / V_{jh} = 1 and N^* / ($A_g f_c^{'}$) = 0 if the shear stress ratios v_{jh} / $f_c^{'}$ are the same in both joints. Assuming that crushing of the diagonal compression field in the centre of joint panel leads to failure, it may be reasonable to say that the joint with horizontal reinforcement can sustain approximately 2.8 times the stress ratio v_{jh} / $f_c^{'}$ of the unreinforced joint if the same ratio f_{cs} / $f_c^{'}$ is to be attained. According to this rationale it is possible to relate the shear stress ratio v_{jh} / $f_c^{'}$ of a beam-column joint with given values of N^* / ($A_g f_c^{'}$) and V_{sh} / V_{jh} to the shear stress ratio v_{jh} / $f_c^{'}$ of a joint with N^* / ($A_g f_c^{'}$) = 0 and V_{sh} / V_{jh} = 1, so that both joints have equal stress ratios f_{cs} / $f_c^{'}$. This transformation can be achieved using factor K_{pv} shown in the vertical axis at the right hand side of Fig. 4. Factor K_{pv} is defined as:

$$K_{pv} = (v_{jh,e} / f_c^{'}) / (v_{jh} / f_c^{'}) \quad (1)$$

Where $v_{jh,e}$ is referred to as the horizontal joint shear stress of the equivalent joint, which, by definition, has N^* / ($A_g f_c^{'}$) = 0 and V_{sh} / V_{jh} = 1.

DATABASE REDUCTION

Data from cyclic reversed load tests on beam-column joint assemblies was collected and reduced. The database excluded tests in which beam-column joints failed prior to yielding of the beam longitudinal reinforcement. Tests in beam-column joints reinforced with hoops without a well-defined yield plateau were also excluded.

Furthermore, joint assemblies incorporating lateral beams that were not loaded were not considered either. The joint shear stress of beam-column assemblies was transformed to an equivalent stress using Eq.(1). The displacement capacity of the test units was defined equal to the displacement associated with 10 percent strength degradation measured in the lateral load - lateral displacement response envelope. The usefulness of the database would have been very limited if the normally accepted 20 percent strength degradation concept [Park, 1989] had been used in the study.

Figure 5 plots the rotational ductility capacity $\mu_{\theta,}$ versus equivalent joint shear stress ratio $v_{jh,e} / f_c^{\prime}$ of the test assemblies compiled in the database. The rotational ductility is defined similarly to the displacement ductility [Park, 1989]. The difference between both definitions is that the rotational ductility does not consider the elastic component of the column displacement [Lin et al. 1997; Lin, 1999]. Figure 5 shows a clear trend. Beam-column joint failures occur after beam flexural yielding if the equivalent joint shear stress ratio exceeds $v_{jh,e} / f_c^{\prime} = 0.3$. For smaller equivalent joint shear stress ratios failure takes place in the beams. The relationship between the stress ratio f_{cs} / v_{jh} and the rotational ductility $\mu_\theta$in assemblies failing in the joint has a physical explanation. Yield of the deformed beam longitudinal reinforcement anchored in the joint penetrates gradually with the ductility imposed in the plastic hinges. Also, the horizontal joint reinforcement begins to yield to sustain the diagonal compression field. Yielding of the horizontal reinforcement can become unrestricted, particularly if this reinforcement is characterised by a well-defined yield plateau. A consequence of unrestricted yielding is dilation of the concrete in the plane of the joint, which leads to the reduction in the strength of the diagonal compression field.

HORIZONTAL JOINT REINFORCEMENT

Design Charts

The failure criteria shown by the bi-linear trend in Fig. 5 can be used to develop design charts for different ductility or interstorey drift levels, depending on the design criteria [Lin, 1999]. For example, in ductility based design of frames designed to form beam sidesway mechanisms, rotational ductility demands of approximately 7.7 and 3.7 may be expected for fully ductile or limited ductility response, respectively. The equivalent joint shear stress ratios these ductility levels are 0.3 and 0.52 when using the 95 percent confidence limit line shown in Fig. 5.. The design charts plotted in Fig. 6 are found substituting these equivalent joint shear stress ratios and the values of K_{PV} shown in Fig. 4 in Eq. 1.

There are three distinct regions in the two charts depicted in Fig. 6. Firstly, the amount of joint reinforcement is rather insensitive to axial compression when the column axial load ratio ranges between 0 and 0.1. Secondly, when the column axial load ratio increases from 0.1 to 0.3 the required quantity of reinforcement decreases. Thirdly, the amount of reinforcement increases with an increase in axial load for column axial load ratios greater than 0.3, with large amounts apparently needed for joints with moderate to large stress ratios $v_{jh} / f_c^{\prime}$ when $N^* / (A_g f_c^{\prime}) > 0.4$. It is noted here that there is a lack of experimental work in joints subjected to high axial loads to endorse the observed trend. The charts can be used to write simple and practical design recommendations. For example, the uncertainty in evaluating the column axial load in columns subjected to earthquake loading suggests that design recommendations should be somehow desensitised from this parameter.

Design Recommendations

The horizontal shear force in an interior beam-column joint can be expressed as the sum of three different transfer mechanisms,

$$V_{jh} = V_c + V_N + V_{sh} \tag{2}$$

where V_{jh} is the horizontal joint shear force, V_c is the shear force carried by the concrete, V_N is the component of the shear force carried by the column axial load and V_{sh} is the shear force carried by the horizontal joint reinforcement.

For joints of frames designed for fully ductile response,

The force carried by the concrete is,

$$V_c / V_{jh} = 1 / \{600 (v_{jh} / f_c^{\prime})^3\} \leq 1 \tag{3a}$$

The force carried by the column axial load is,

$$V_N / V_{jh} = 0 \quad \text{when} \quad N^* / (A_g f_c^{\prime}) \leq 0.1 \tag{3b}$$

when $0.1 < N^* / (A_g f_c^{\prime}) \leq 0.4$ V_N / V_{jh} is the lesser of,

$$V_N / V_{jh} = 1.6 \{ N^* / (A_g f_c^{\prime}) - 0.1\} \tag{3c}$$

and

$$V_N / V_{jh} = 0.15 \tag{3d}$$

The horizontal joint reinforcement V_{sh} / V_{jh} can be determined from Eq. 2 once V_c / V_{jh} and V_N / V_{jh} have been found. Then, the amount of horizontal joint reinforcement A_{sh}, can be obtained as,

$$A_{sh} = V_{sh} / f_{yh} \tag{5}$$

where f_{yh} is the yield strength of the reinforcement.

It is recommended the following requirements be satisfied,

$$V_{sh}/V_{jh} \geq 0.4 \text{ and } v_{jh} \leq 0.25 f_c^{\prime} \tag{4}$$

For interior beam-column joints of frames designed for limited ductility response it is recommended $V_{sh} = 0.4 V_{jh}$ and $v_{jh} \leq 0.25 f_c^{\prime}$.

CONCLUSIONS

1. A behavioural model based on the lower bound theorem of plasticity is proposed for evaluating the internal force flow within an interior beam-column joint panel. The model can be used to improve the understanding of the transfer mechanisms in joints.

2. The model was used to perform a parametric analysis to evaluate the relative importance of variables effecting the flow of internal forces in the joint panel. The main variables found to significantly influence the internal force flow were the column axial load and the horizontal joint reinforcement.

3. The analytical model was calibrated with results of tests on interior beam-column joints that showed yielding of the beam longitudinal reinforcement at the column faces and either failed in the joint panel or in the beam plastic hinges. The ductility demand in the beams was found to influence the strength of the diagonal compression field.

4. By evaluating the demand and capacity, design charts and equations were proposed for the seismic design of interior-beam column joints.

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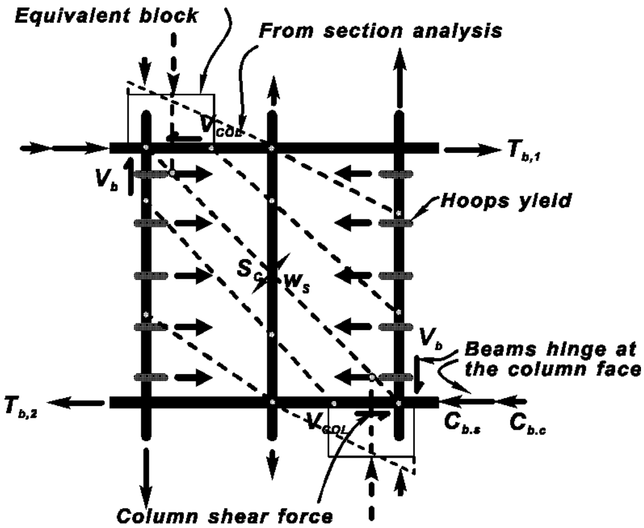
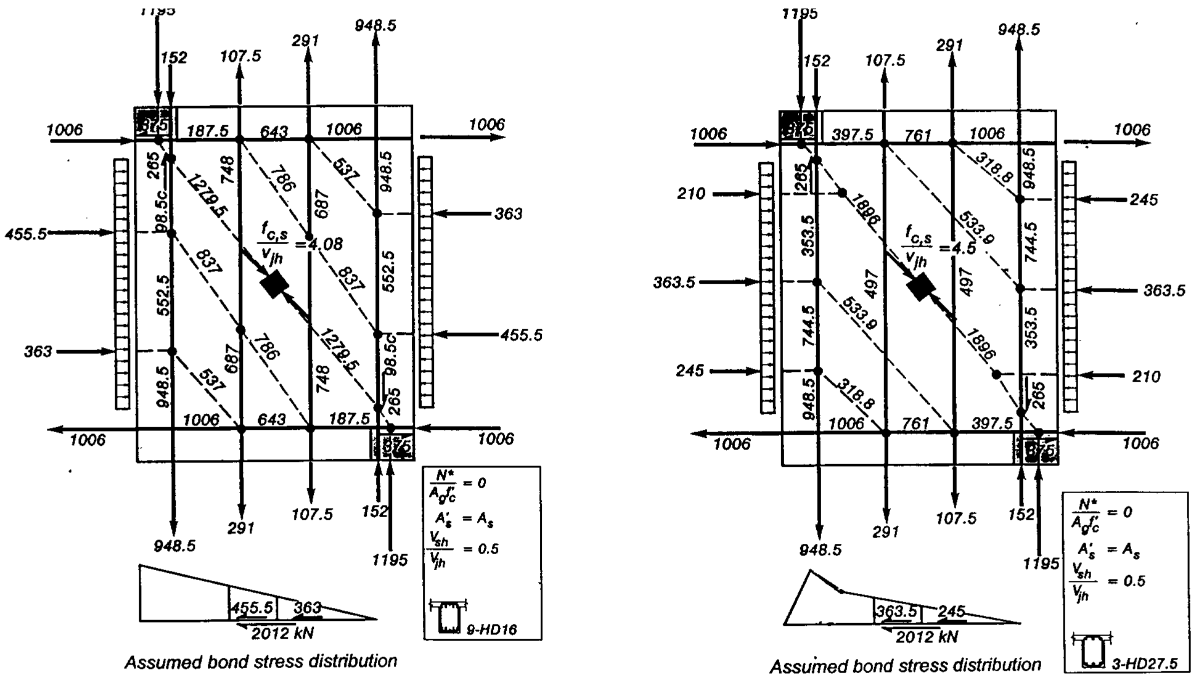


$$f_{C,s} = \frac{S_C}{w_s\ b_j}$$

Figure 1: Model assumptions



(a) Large h_c / d_b ratio (b) Small h_c / d_b ratio

Figure 2:Effect of beam bar bond stress distribution on the stress ratio $f_{c,s} / v_{jh}$

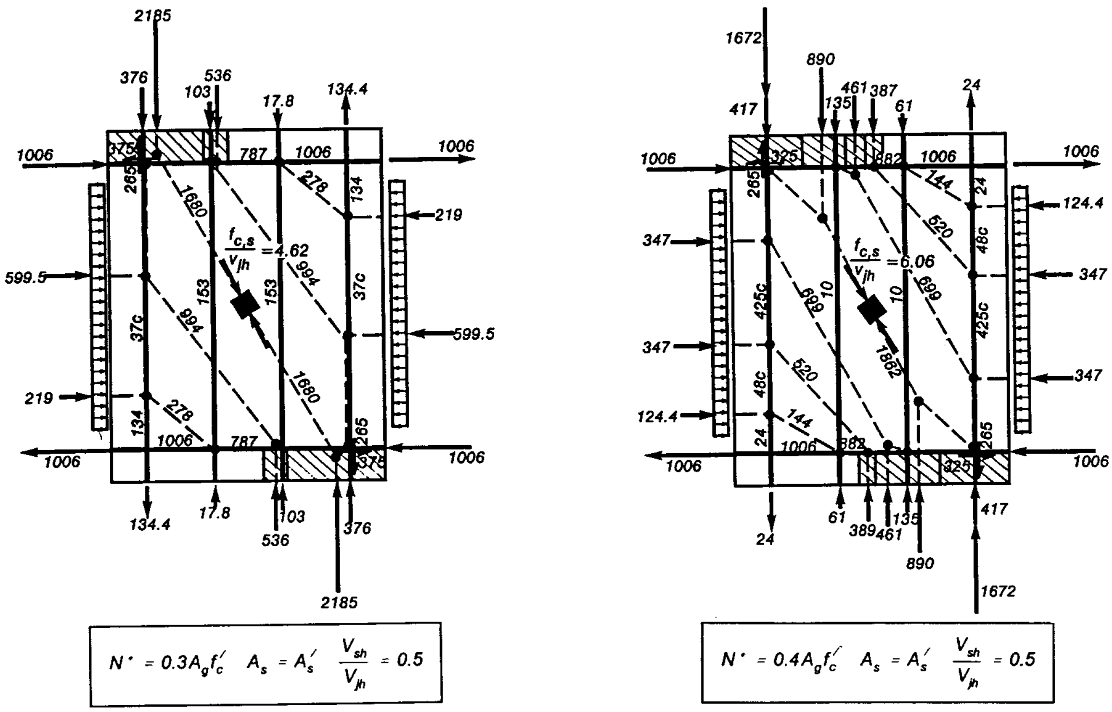


Figure 3:Effect of column axial load on the stress ratio $f_{c,s} / v_{jh}$

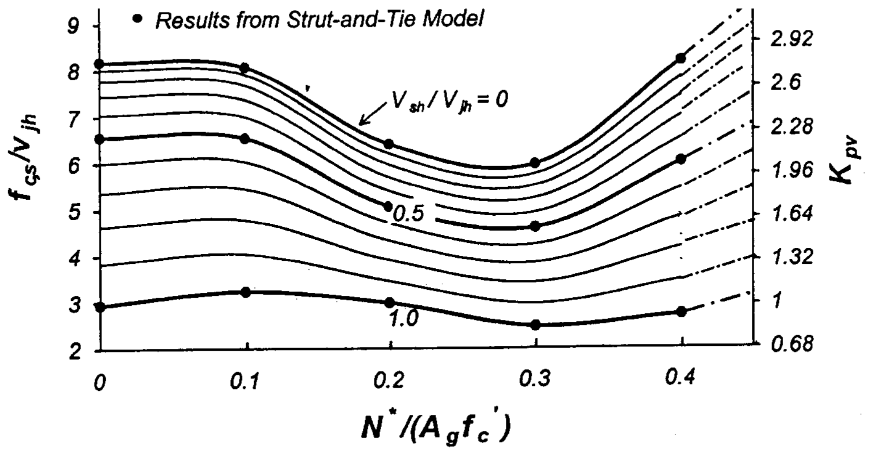


Figure 4: Combined effect of ratios $N^*/(A_g f_c')$ and V_{sh}/V_{jh} on the stress ratio $f_{c,s}$ / v_{jh}

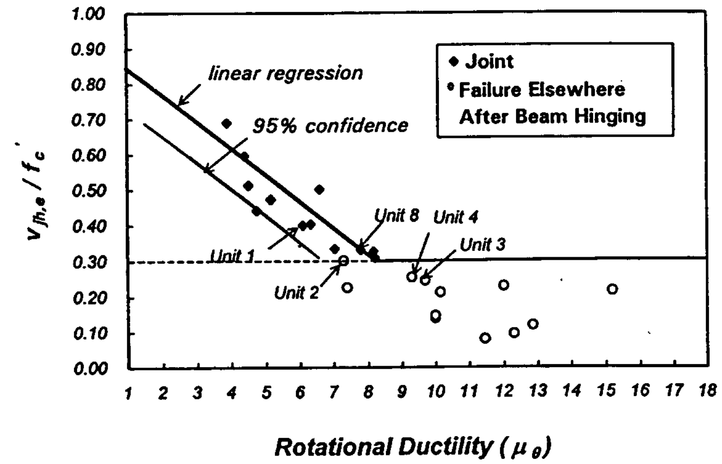
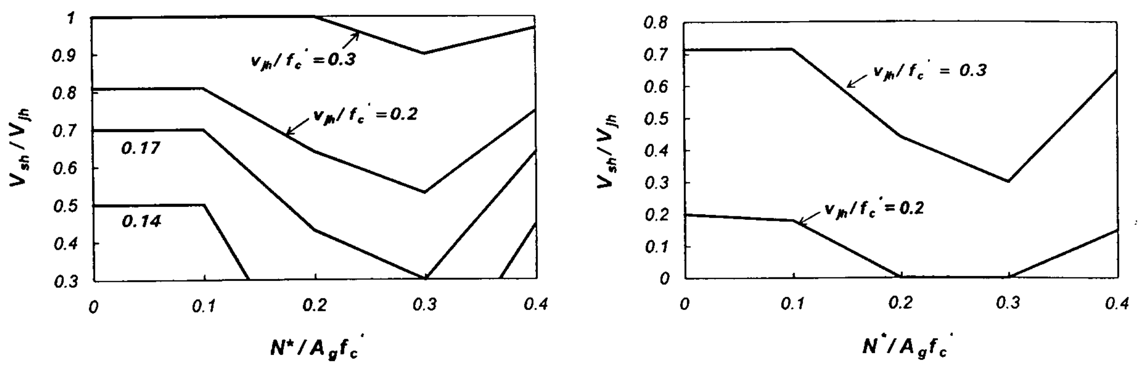


Figure 5: Evaluation of test results



(a) Joints of ductile frames

(b) Joints of limited ductility frames

Figure 6: Horizontal joint reinforcement design chart