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EVALUATION METHODS FOR REINFORCED CONCRETE COLUMNS AND CONNECTIONS

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Abstract

Damage to reinforced concrete frames not meeting current seismic code regulations has been prevalent in recent earthquakes. Performance of reinforced concrete frame buildings in past earthquakes reveals common failure modes: shear failure and/or splice failure of columns, shear failure of beam-column joints or pullout of reinforcement embedded in beam-column joints. The following paper presents methods to evaluate the strength of reinforced concrete columns and connections with deficient details. Recent experimental and analytical research efforts at the University of California at Berkeley have focused on methods to evaluate concrete frames vulnerable to damage in an earthquake. Evaluation methods to assess the strength of reinforced concrete columns and connections are validated using experimental research results.

Keywords

Reinforced concrete, columns, beam-column joints, retrofit, evaluation, rehabilitation, splices, shear strength

Introduction

Reinforced concrete buildings designed according to older code provisions have been found to be especially vulnerable to earthquake damage. Where current code regulations have stringent detailing requirements to ensure ductile behavior, previous regulations have been primarily strength based. Although new design permits economical construction of well-detailed components, the construction cost of upgrading schemes may be prohibitive. An accurate assessment of the system capacity may be required for economical reasons.

The paper presents methods to evaluate the strength of columns and beam-column connections found in older reinforced concrete building construction. The methods included were verified using results from recent experimental efforts at U.C. Berkeley as well as other research institutions. Methods to evaluate the shear strength and lap splice capacity of reinforced concrete building columns are presented in light of recent column tests. Strength of connections is evaluated considering bar pullout and joint shear.

Details

In existing, pre-1970's construction, it is common to find column longitudinal reinforcement spliced just above the joint where maximum moments develop. Splice lengths and transverse reinforcement along the splice were often calculated assuming the splice acted only in compression; the resulting splice tensile strength and ductility are commonly inadequate for expected loadings. Column longitudinal reinforcement may be poorly distributed around the perimeter of the column core. Transverse reinforcement was often sized to resist code-specified shear forces and may be inadequate to resist the shear corresponding to development of column or beam flexural plastic hinges. It is not uncommon for beams bottom longitudinal reinforcement to terminate a short distance into the joint, creating the possibility of bar slip (or pullout) under moment reversals. Column bars may be poorly distributed around the joint perimeter, and may be spliced just above the joint. Finally, there may be minimal transverse reinforcement in the joint, or none at all. Other potential problems such as eccentric joints may also be found.

Materials

Evaluating the behavior of existing reinforced concrete construction requires evaluation of *in situ* material strengths. Assumed material strength values should be realistic yet conservative estimates of expected values. Longitudinal reinforcement yield strength commonly may be as low as the specified yield strength; however, yield strengths exceeding the minimum specified strength by as much as 20 percent of the nominal value also are not uncommon. For members subjected to inelastic moment reversals, high yield strength combined with strain hardening may result in stresses as high as $1.5f_y$ (f_y is defined as the specified or nominal yield strength). Concrete material strengths vary widely relative to design values. With well-compacted, well-cured concrete, compressive strengths usually exceed design values at early ages and continue to increase with time. In other cases, substandard concrete will be found.

Columns

Response and failure of a reinforced concrete column in a building frame under reversed cyclic loading may be controlled by combined axial load and flexure, shear, splice failure, or a combination of these. An experimental program at the University of California at Berkeley has studied these aspects for deficient building columns (Lynn and Moehle). Eight columns were constructed at full scale with an 18-in. (46-cm) square cross-section and 10-ft (3-m) clear height. The columns were reinforced with Grade 40 (275 MPa) steel, either eight # 8 bars (25-mm) or eight # 10 (32-mm) bars longitudinally with #3 (1-mm) Grade 40 (275 MPa) perimeter hoops. Ties used in the first six specimens were square hoops with an 18-in. spacing. Specimens 7 and 8 were detailed with diamond ties spaced at 12 inches. Lap splices, used in three of the eight columns, have a length of 20 longitudinal bar diameters. The loading included axial load plus reversed cyclic lateral load with zero imposed rotation at the column ends. Details of all columns in the test series are provided in Table 1.

Flexural-axial strength of column sections with light transverse reinforcement can be calculated using standard ACI methods with expected material strengths, with direct consideration of material overstrength, strain hardening, or understrength is sufficient. Table 1 presents flexural strengths for the columns tested by Lynn and Moehle computed according to ACI 318-95.

Figure 2 displays results for specimen 2 (Lynn and Moehle). The column was detailed with lapped longitudinal bars. For Grade 40 bars, 20-bar diameter laps, and widely-spaced ties, the bars are barely able to develop yield, and rapidly lose capacity following yield. When the lap fails, moment capacity at the lap reduces to a value corresponding approximately to the product of the axial load and half the section depth. This failure may transform an otherwise strong-column/weak-beam connection into a weak-column/strong-beam connection.

For columns with short, unconfined lap splice lengths, the cover concrete controls the splice capacity. The stress capacity of the splice may be computed according to equation proposed by Orangun *et al.*, as follows:

$$f_s = \frac{4ul_s}{d_b} \leq f_y \; . \qquad (1)$$

where the bond strength, u, is determined using Equation 2.

$$u = (1.22 + 3.23C/d_b + 53d_b/l_s)\sqrt{f_c'} \text{ with } C/d_b \leq 1.5, \text{ with } f_c' \text{ in psi.} \tag{2}$$

This formulation has been verified for nominal steel strengths of 60 ksi (400 MPa) or less, and specified concrete strengths not exceeding 5 ksi (30 MPa).

Post-yield behavior of a lap splice is strongly dependent on the amount and arrangement of the transverse reinforcement. According to Sivakumar, *et al.* a well-confined lap splice has transverse steel at a spacing not exceeding s_{max}, with s_{max} defined as follows:

$$s_{max} = \frac{25l_s\sqrt{A_{tr}}}{f_y d_b^2} * \frac{m}{n} \tag{3}$$

where the ratio m/n is 1 for circular sections. When Equation (3) is not satisfied, it is likely that the stress capacity will degrade with continued cycling. When Equation (3) is satisfied, the post-yield behavior may allow excursion into the inelastic range without rapid degradation.

Experimental details for five specimens with inadequate lapped splices are shown in Table 2. For all columns, failure corresponded to loss of lap splice capacity. Results for the five specimens using Equations 1, 2 and 3 is also included in the table. Using Equation 2 for a typical building column, represented by Specimen 2 by Lynn and Moehle, results in a bond strength of $10\sqrt{f_c'}$, psi.

Shear strength of a reinforced concrete column varies with concrete strength, transverse reinforcement, axial load, load history and flexural ductility demand. Shear strength expressions used for the design of new building columns tend to be unnecessarily conservative for existing construction where the engineer does not have the opportunity to place copious amounts of transverse reinforcement at a reasonable cost. Alternative expressions may be desirable for existing construction.

Figure 3 compares the observed variation of shear strength as a function of displacement ductility demand using experimental results from Lynn and Moehle. All columns failed in an apparent shear mode (as indicated by crack patterns) following flexural yield. The plot shows the normalized shear as a function of the displacement ductility, both at time of failure. The displacement ductility is defined as the ratio of the displacement corresponding to a 20% reduction in strength to the displacement corresponding to first yield of the longitudinal bars. The data indicate that shear strength is reduced for increased displacement ductility demand. On the basis of the data shown in Figure 3, as well as other data, the following model for shear strength is proposed. The equation is appropriate for building columns with an aspect ratio exceeding 2.5.

$$V_n = 3.5\left(q + \frac{N_u}{f_c' A_g}\right)\sqrt{f_c'}A_e, \text{(psi) with } 1 \geq q = (4 - \mu_\delta)/3 \geq (1/3.5) \tag{4}$$

Typical tie spacings in existing buildings column exceed d/2, rendering the shear strength normally associated with transverse confinement (ACI-318) negligible. Equation 4 is appropriate for columns with widely-spaced ties. For columns with a larger amount of transverse reinforcement, the following expression from Aschheim and Moehle (1992) may be more appropriate.

$$V_n = V_c + V_s,\ V_c = 3.5\left(k + \frac{N_u}{2000A_g}\right)\sqrt{f_c'}A_e, \text{(psi)},\ V_s = \frac{mA_{tr}f_y d}{s\tan 30} \tag{5}$$

with $1 \geq k = (4 - \mu_\delta)/3 \geq 0$

Values given by the previous expressions should be interpreted with caution. The displacement ductility term, μ_δ, is difficult to assess, in the laboratory and more so in existing buildings. Furthermore, the slope of the line representing the relationship between column shear strength and displacement ductility demand is dependent on the load history; a larger number of cycles at the same displacement ductility demand is likely to result in a steeper slope and vice versa. Both issues should be considered when evaluating column shear strength.

The preceding expressions have been developed for slender columns. Studies show these expressions are unnecessarily conservative for columns with an aspect ratio of less than 2.5. The following expressions from Umehara and Jirsa give reasonable correlation with observed shear strengths for columns with aspect ratio less than 2.5.

$$V_n = \left(11 - 3\frac{a}{d}\right)A_c\sqrt{f_c^{'}} + \frac{0.2N_u}{a/d}, \text{psi} \; ; \; \frac{0.2N_u}{a/d} \le \frac{160A_g}{a/d}, \; 1 \le a/d \le 2.5 \qquad (6)$$

Beam-Column Connections

Embedded Bar Strength

Strength of a beam-column connection may be limited by pullout of discontinuous bottom beam reinforcement. To determine if the bar embedment length is adequate, a procedure adopting guidelines developed by Eligehausen *et al.* is used. In the adaptation, the stress in the column longitudinal reinforcement is used as an indicator of the transverse stress field acting on the embedded bar. For zero column reinforcement tensile stress, the maximum pullout strength is obtained corresponding to shearing failure of the concrete surrounding the bar. For high column reinforcement stresses, a minimum pullout strength is obtained. The procedure is as follows:

1. Following the recommendations of Eligehausen *et al.*, the maximum and minimum strengths for normal strength concrete and for bars that are #10 or less are:

$$F_{min} = 5\sqrt{f_c^{'}\,\text{psi}}\,(\pi d_b l_s) \; ; \; F_{max} = 30\sqrt{f_c^{'}\,\text{psi}}\,(\pi d_b l_s) \qquad (7)$$
$$\text{with } F_{min} < F_{max} \le F_y$$

   where F_{max} is less than or equal to the expected yield strength.
2. The flexural strengths of the beam, M_{bmin} and M_{bmax}, corresponding to development of F_{min} and F_{max}, are determined.
3. The column flexural demands, M_{co} and M_{c1},, are computed. Using the Eligehausen *et al.* recommendations, development of M_{bmax} in the beam corresponds to no tensile stress in the column longitudinal bars. Development of M_{bmin} in the beam corresponds to a tensile stress of 43 ksi in the column longitudinal bars. Therefore, M_{co} corresponds to $f_{sc} = 0$ ksi and M_{c1} corresponds to $f_{sc} = 43$ ksi.
4. Assuming the flexural demand is distributed according to the relative stiffnesses of the columns adjacent to the connection, the maximum and minimum flexural demands in the column, M_{cmin} corresponding to M_{bmin} and M_{cmax} corresponding to M_{bmax}, are determined.
5. The following table can be constructed and the two series plotted (Figure 4):

X axis	Series 1	Series 2
M_{bmin}	M_{c1} (43 ksi)	M_{cmin}
M_{bmax}	M_{co} (0 ksi)	M_{cmax}

   The intersection point of the two series is the beam flexural strength, M_b, corresponding to bond failure of the embedded bar.

Table 3 presents experimental data for joints with embedded bars. The embedment length, material strength and experimental results are presented. The flexural strength, M_b, computed using the above procedure gives reasonable correlation with the experimental values.

Shear Strength

Several researchers have reported behavior of interior and exterior connections representative of those found in pre-1970's concrete construction. A commonly reported index is the nominal joint shear stress before onset of joint failure. This measure of joint capacity must be viewed cautiously. Joint shear strength appears to depend not only on joint size, geometry, materials, and reinforcement quantity, but also on bond conditions within the joint and flexural ductility levels of the adjacent framing members. For example, a joint shear strength measured in a test in which the framing members did not yield may not be applicable for an identical joint in which the framing members are yielding. More general relations between joint shear strength and component ductility levels are desirable but not yet available for existing construction details. Modern beam-column connections in ductile moment resisting frames are required to remain intact even after flexural yielding of adjacent members. Since most upgrading schemes require ductile retrofit of adjacent members, joint shear strength values reported herein are for joints failing in shear following flexural yielding only.

Figure 5 presents data on interior joints gathered by Otani. These data suggest that interior joint shear strength is sensitive to small changes in transverse reinforcement, but strength does not increase significantly for transverse reinforcement ratios above about 0.003 (relevant data labeled with triangles). Similar results were reported by Kurose *et al.* as shown in Figure 6 (relevant data labeled with squares) (The mechanical joint lateral reinforcement ratio of 3 is approximately equivalent to a reinforcement ratio of 0.003 for typical material strengths found in existing construction). Figure 7 presents data on exterior joints reported by Kurose *et al.* (relevant data labeled with squares). Note that for exterior joints, ACI 318-95 prescribes a joint shear strength of $12\sqrt{f_c'}A_j$. All exterior joints (without transverse beams) failing at a joint shear demand less than that prescribed by ACI had a deficient amount of transverse steel.

On the basis of the preceding information, nominal joint shear strength can be expressed as:

$$V_n = \lambda\gamma\sqrt{f_c'}A_j, \text{psi} \tag{8}$$

in which λ= 0.75 for LWC or 1 for NWC, and γ and A_j are as defined below.

Value of γ	Interior joint		Exterior joint		Knee joint
ρ''	With transverse beams	Without transverse beams	With transverse beams	Without transverse beams	
<0.003	12	10	8	6	4
≥0.003	20	15	15	12	8

Effective joint area A_j is defined according to ACI 318-95 for concentric joints. Joint shear strength values for exterior joints without transverse beams may overestimate the joint shear strength capacity of joints with high flexural ductility demand. Use of joint shear strength values without further experimental verification should be done so with caution.

Acknowledgments

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References

ACI-318 (1995), Building Code Requirements for Reinforced Concrete, American Concrete Institute

Aboutaha, R.S.; Engelhardt, M.D.; Jirsa, J.O.; Kreger, M.E. (1993), Seismic Strengthening of R/C Columns Using Steel Jackets, ASCE Structures Congress 1993, Irvine CA

Aschheim, M.A. and Moehle, J.P. (1992), Shear Strength of Deformability of R/C Bridge Columns Subjected to Inelastic Cyclic Displacements, U.C. Berkeley, Report No. UCB/EERC-92/04, April 1992

Beres, A.; El-Borgi, S.; White, R.N.; Gergely, P. (1992), Experimental Results of Repaired and Retrofitted Beam-Column Joint Tests in Lightly Reinforced Concrete Frame Buildings, NCEER-92-0025

Chai, Y. H.; Priestley, M.J.N.; Seible, F. (1991) Seismic Retrofit of Circular Bridge Columns for Circular Flexural Performance, *ACI Structural Journal*, Vol. 88 no. 5, Sept. 1991

Eligehausen, R.; Popov, E.P.; Bertero, V.V. (1983) Local Bond Stress-slip Relationships of Deformed Bars under General Excitations, U.C. Berkeley, Report No. UCB/EERC-83/23, Oct. 1983

Kurose, Y.; Guimaraes, G.N.; Liu, Z.; Kreger, M.E.; Jirsa, J.O. (1988), Study of R/C Beam-Column Joints under Uniaxial and Biaxial Loading, University of Texas at Austin, PMFSEL Report 88-2

Lowes, L.N.; Moehle, J.P. (1995), Seismic Behavior and Retrofit of Older Reinforced Concrete T-Joints, U.C. Berkeley, UCB/EERC - 95/09, Sept. 1995

Lynn, A.C.; Moehle, J.P. (1996) Evaluation of Existing Reinforced Concrete Building Columns, U.C. Berkeley, Internal Document

Orangun, C.O.; Jirsa, J.O.; Breen, J.E. (1977) A Reevaluation of Test Data on Development Length and Splices, *ACI Structural Journal*, March 1977, pp. 114-122

Otani, S. The AIJ Proposal of Ultimate Strength Design Requirements for R/C Buildings with Emphasis on Beam-Column Joints, Design of Beam Column Joints for Seismic Resistance, ACI SP-123

Pessiki, S.P.; Conley, C.H.; Gergely, P.; White, R.N. (1990), Seismic Behavior of Lightly Reinforced Concrete Column and Beam-Column Joint Details, NCEER-90-0014

Priestley, M.J.N.; Seible, F.; Chai, Y.H.; Wong, R. (1992), Santa Monica Viaduct Retrofit - Full-Scale Test on Column Lap Splice with #11 [35mm] Reinforcement, UCSD Report No. SSRP-92/08, Sept. 1992

Sivakumar, B.; Gergely, P.; R.N. White (1983), Suggestions for the Design of R/C Lapped Spliced for Seismic Loading, *Concrete International*, February 1983, pp. 46-50

Soyer, C.; Moehle, J.P. (1992), Behavior of R/C Cap Beam-Column Exterior Bridge Joints, U.C Berkeley, 299 Report, April 1992

Umehara, H.; and Jirsa, J. (1984), Short Rectangular R/C Columns Under Bidirectional Loadings, *Journal of Structural Engineering*, Vol. 110, no. 3, March 1984

Valluvan, R.; Kreger, M.E.; Jirsa, J.O. (1993) Strengthening of Column Splices for Seismic Retrofit of Nonductile Reinforced Concrete Frames, *ACI Structural Journal*, Vol. 90, no. 4, July 1993

List of Symbols

a/d - column aspect ratio
A_c - area of core cross-section
A_e - area of effective cross-section
A_g - area of gross cross-section
A_j - effective joint area (ACI-318)
A_{tr} - area of transverse reinforcing bar
C - concrete cover
d_b - bar diameter
f'_c - concrete compressive strength
f_s - lap splice strength
f_{sc} - tensile stress in column longitudinal bars
f_y - yield strength of longitudinal steel
l_s - lap splice length
m - number of transverse reinforcing bars
M_{exp} - experimental flexural strength
μ_δ - displacement ductility
n - number of spliced longitudinal bars
n_s - number of cycles sustained
N_u - axial load
ρ'' - transverse reinforcement ratio
s - actual tie spacing
s_{max} - maximum tie spacing
u - bond stress
V_c - shear strength attributed to concrete
V_n - shear strength of concrete section
V_s - shear strength attributed to steel
V_u - experimental shear strength

Tables and Figures

I.D.	f'_c	d_b	s	l_s	N_u	V_u	Δ_u	μ_δ	M_{exp}	M_{ACI}
1	3.71	1.25	12	no splice	113.4	61	1.91	1.60	3539	3604
2	3.71	1.25	12	25 in.	113.4	60	2.03	1.44	3481	3604
3	4.80	1.00	12	no splice	113.4	54	2.03	3.75	3133	2718
4	4.80	1.00	12	20 in.	113.4	52	2.16	4.94	3017	2718
5	3.70	1.00	12	no splice	340.0	71	1.27	1.20	4119	3364
6	4.00	1.25	12	no splice	340.0	76	1.07	1.43	4410	4258
7	4.00	1.25	18	no splice	340.0	80	1.91	1.20	4642	4258
8	3.70	1.25	18	25 in.	340.0	85	1.91	1.20	4932	4134

Table 1 Specimens Tested by Lynn and Moehle

Reference (I.D.)	f'_c	f_y	d_b	s	l_s	f_s actual	f_s Eqn. 2	f_{sact}/f_{scomp}	s_{max}	n_s
Lynn (2)	4.70	50	1.00	18	20	50	50	1.00	2	2
Aboutaha (1)	2.85	70	0.98	16	24	53	52	1.01	2	1
Priestley (1)	4.06	45	1.38	3.5	30	45	45	1.00	4	5
Chai (1)	5.54	46	0.75	5	15	46	46	1.00	3	
Valluvan (1)	3.50	70	0.75	12	18	42	44	0.96	3	1

Table 2 Splice Strength

Reference	f'_c	f_y	d_b	l_{de}	M_{exp}	M_b	M_{exp}/M_b
Pessiki (7)	3.37	69.4	1.00	6.0	1121	841	1.33
Pessiki (8)	3.37	69.4	0.75	6.0	1121	1104	1.01
Beres (4)	3.37	69.4	1.00	6.0	752	671	1.12
Lowes (1)	5.10	48	0.88	17.5	1472	1488	0.99
Soyer (1)	6.16	46	0.75	15.0	M_p	M_p	1.00

Table 3 Embedded Bar Strength

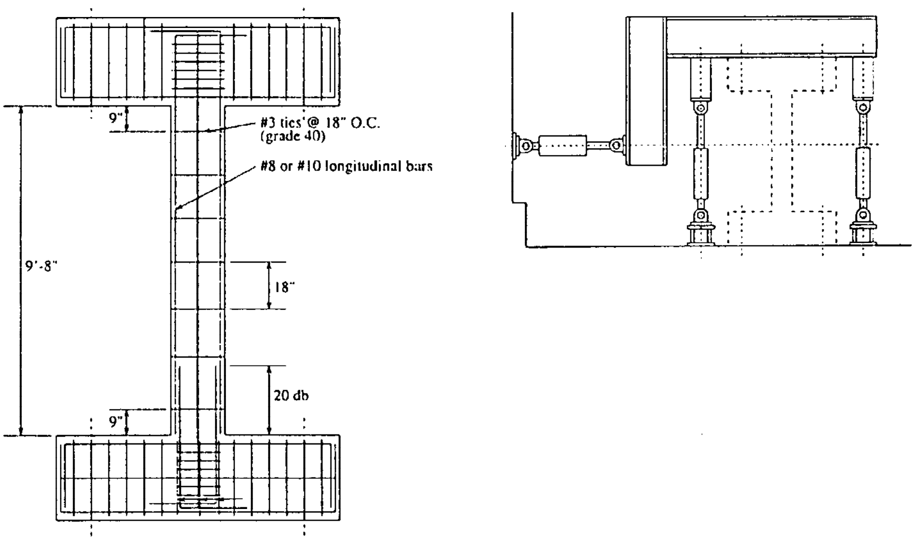


Figure 1 Columns Tested by Lynn and Moehle

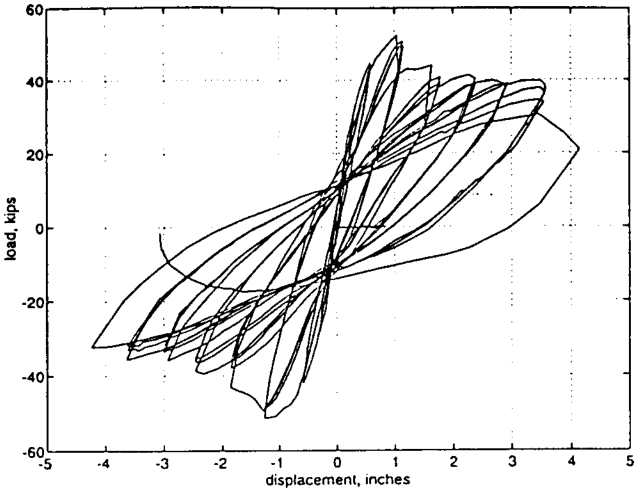


Figure 2 Response of Column with Spliced Bars

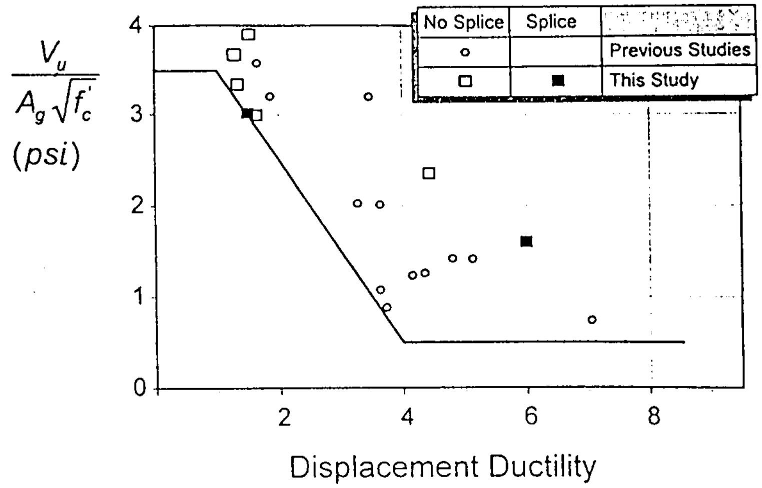


Figure 3 Experimental Column Shear Strength

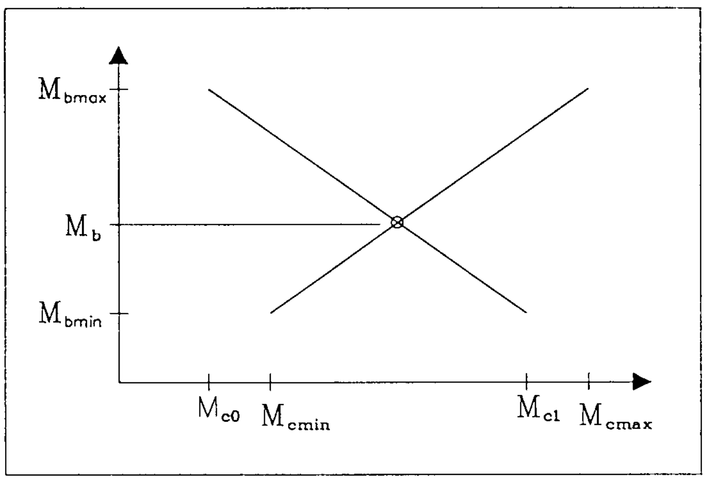


Figure 4 Graphical Computation of M_b

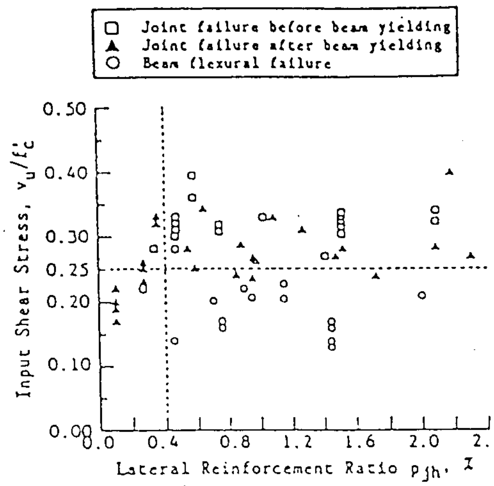
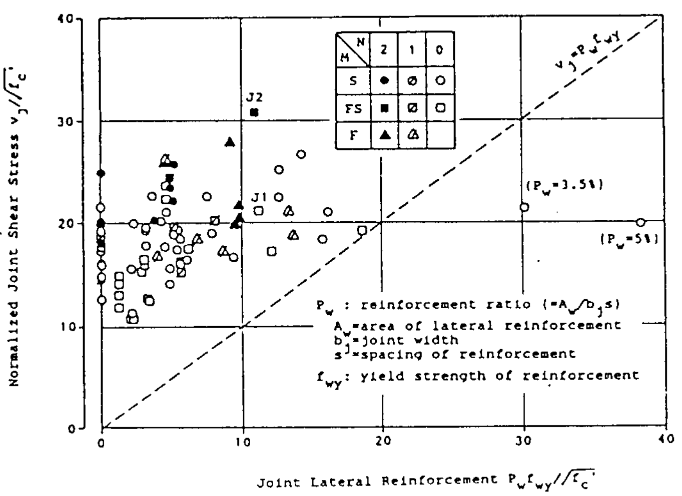


Figure 5 Interior Joint Shear Strength (Otani)



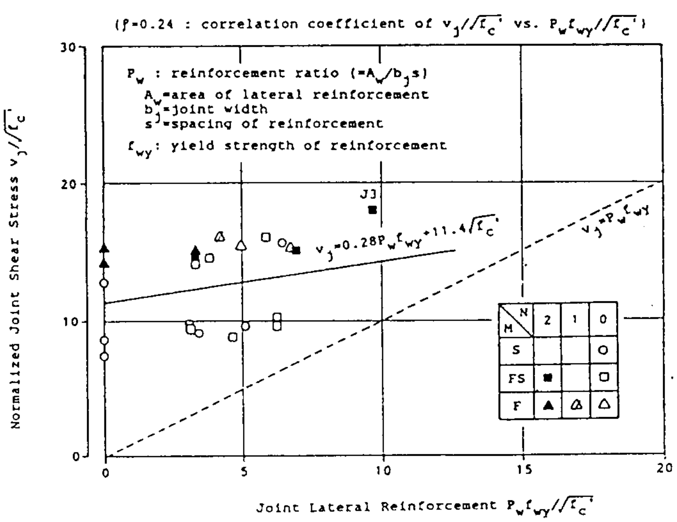


Figure 7 Interior Joint Shear Strength Kurose *et al.*

Figure 8 Exterior Joint Shear Strength Kurose *et al.*