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철근콘크리트 특수모멘트골조의 보-기둥 접합부 실험체의 내진성능평가

Seismic Evaluation of Beam-Column Joint Specimens of RC Special Moment Frames

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요 약

본 연구는 철근콘크리트 특수모멘트골조(SMF)내 보-기둥 접합부의 비탄성 회전능력에 대한 연구결과를 조사한 것이다. 모든 실험체들은 ACI 318-02에 정의된 설계 및 세부지침에 따라 특수모멘트골조로 분류되었다. AISC(2002)기준에서 모멘트골조의 접합부에 대한 내진성능의 만족 기준을 이용하여 보-기둥 접합부를 평가하였다. 총 39개의 실험체들에 대해 자세하게 조사되었다. 특수모멘트골조에 대한 내진설계기준을 만족하는 대부분의 접합부들은 3%의 소성회전까지 휨강도의 심각한 감소 없이 연성이 유지되었다. 이는 특수모멘트골조 접합부들에 대한 엄격한 콘크리트 내진설계 규정에 따른 것으로 조사되었다. 접합부내의 횡방향 보의 존재는 보-기둥 접합부의 구속력과 전단에 대한 저항성을 증가시킨 것으로 조사되었다. 접합부 전단응력에 대한 ACI 328-02 제한조건을 만족하는 모든 특수모멘트골조의 접합부들은 요구되는 내진성능을 만족하는 것으로 나타났다.

Abstract

This study summarizes the results of a research project aimed at investigating the inelastic rotation capacity of beam-column joints of reinforced concrete special moment frames. All of the test specimens were classified as special moment frame (SMF), based on the design and detailing requirements of the ACI 318-02 provisions. The acceptance criteria, originally defined for steel moment frame connections in the 1997 edition of the AISC Seismic provisions, were used to evaluate the beam-column joints of the reinforced concrete moment frames. A total of 39 test specimens were examined in detail. Most of the joints that satisfy the design requirements for special moment frame structures were found to be ductile up to a plastic rotation of 3% without any major degradation in strength. This is mainly due to the stringent ACI 318-02 requirements for special moment frame joints. The presence of transverse beams increases confinement and shear resistance of joints, which results in better performance than for joints without transverse beams. All of the SMF connections that satisfy the ACI 318-02 limitations on joint shear stress turned out to meet the acceptance criteria

키워드 : 모멘트골조, 회전성능, 내진성능, 비탄성변형, 접합부 전단강도, 지진공학, 보-기둥 접합부

Keywords* : *Moment Frame, Rotation Capacity, Seismic Performance, Inelastic Deformation, Joint Shear Strength, Earthquake Engineering, Beam-column Joints

1. Introduction

The 1994 Northridge earthquake, which was classified as a moderate earthquake, caused many connections of steel MRF to fail, resulting in significant economic losses for the building owners. One of the most recent efforts to limit the level of damage in structures subjected to earthquakes was initiated by the American Institute of Steel Construction (AISC). The revised "Seismic Provisions for Structural Steel Buildings"[1) (hereafter referred to as the AISC Seismic provisions) are the provisions adopted throughout the U.S. for the design and construction of structural steel and composite structural steel/reinforced concrete (RC) building

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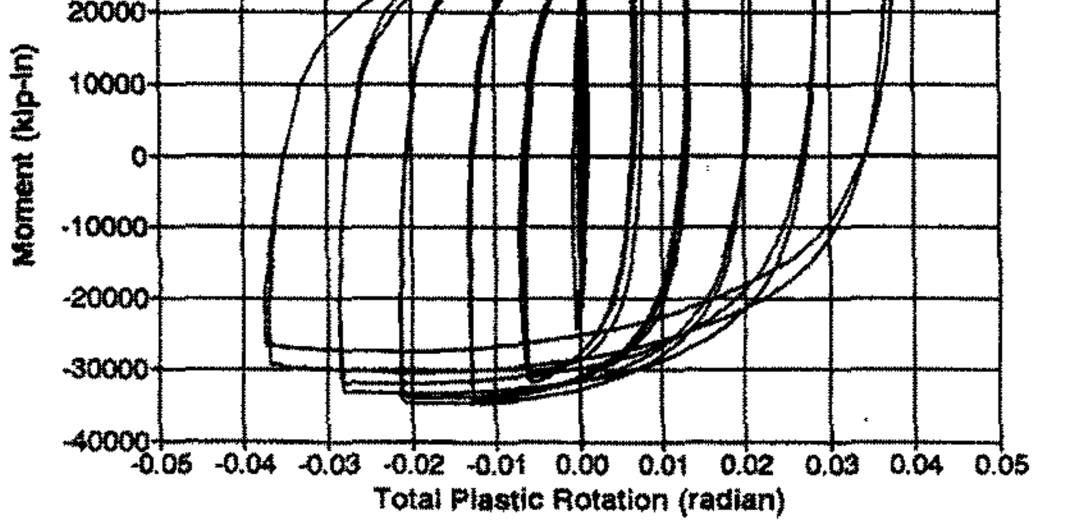
systems in seismic regions. Included are relatively new requirements for beam-to- column connections in special moment frames (SMF), intermediate moment frames (IMF), and ordinary moment frames (OMF), driven mainly by the observed brittle fractures in the beam-column connections of buildings in recent earthquakes. In the AISC Seismic provisions, satisfactory seismic performance levels are achieved by requiring minimum levels of expected inelastic rotation capacity at the joints for the various framing types. Section 9.2a of the 1997 AISC Seismic provisions requires that beam-column joints and connections of SMF systems used as part of the seismic-force-resisting system be able to undergo an inelastic rotation of at least 0.03 radians when subjected to a qualifying cyclic test in accordance with Appendix A. In addition to the minimum level of inelastic rotation, the acceptance criteria also focus on the maximum rate of degradation in strength with inelastic deformations. When this rate of degradation is too large, moment demands from P-delta effects may increase significantly, which can lead to frame instability[1]. Fig. 1 shows the acceptance criteria: the flexural strength Mmax at 0.03 radians must be greater than or equal to M_p, where Mmax is the maximum moment recorded in the tests and M_p is the nominal plastic flexural strength.

The main objective of this study is to evaluate the seismic performance of special and intermediate RC moment frames based on the performance criteria of



〈Fig. 1〉 Acceptable strength degradation according to AISC-97 seismic provisions (AISC, 1997)

the AISC Seismic Provisions. All available test results on RC moment frames were compiled and analyzed as part of this investigation. The significant design variables that are examined and evaluated for RC systems in regards to the performance criteria are joint shear stress, beam-column flexural strength ratio, transverse reinforcement ratio in the joint, and column depth-to-beam reinforcement diameter ratio. This research contributes to a better understanding of the typical behavior of RC moment connections when subjected to seismic loads. The findings and recommendations of this study may be used to modify the response modification factors assigned to RC SMF and IMF in the future seismic codes.

2. Test Results for Special Moment Frames (SMF)

Earthquake resistant designs require a structure to behave with controlled deformation and limited damage, if any, when subjected to a design level earthquake. Since moment frames generally experience fairly large inelastic deformations in some structural members when subjected to an expected maximum ground motions, it is important that the systems possess adequate strength and stiffness to withstand seismic events without collapse. This can be accomplished by following the design and detailing requirements prescribed in the building codes. For RC structures, the appropriate design and detailing requirements are provided in Chapter 21 of the ACI 318-02[2]. These detailing requirements are related to the type of structural framing system, seismic risk level at the site, level of energy dissipation (or toughness), and occupancy of the structure.

The terms special, intermediate, and ordinary are indicative of the degree of required toughness for seismic design. This classification leads to different levels of required detailing that are specified in the ACI 318-02. Cast-in-place SMF structures must conform to Sections 21.2 through 21.5, which contain the most stringent design and detailing requirements. These provisions are meant to provide adequate toughness to ensure sufficient deformation capacity. Among the test specimens considered in this study, a total of 38 specimens were classified as SMF connections based on the requirements specified in Chapter 21 of the ACI 318-02. All the connection specimens were analyzed, and the results were plotted based on the joint shear stress, the column-to-beam flexural strength ratio, and the transverse reinforcement ratio.

All test results reviewed for this study are summarized in Table[13~27]. The test results not included in this study were neglected because either there were insufficient data available (i.e. test results performed in Japan) or tests were performed with

〈Table 1〉 List of beam-column joint connections considered in this study

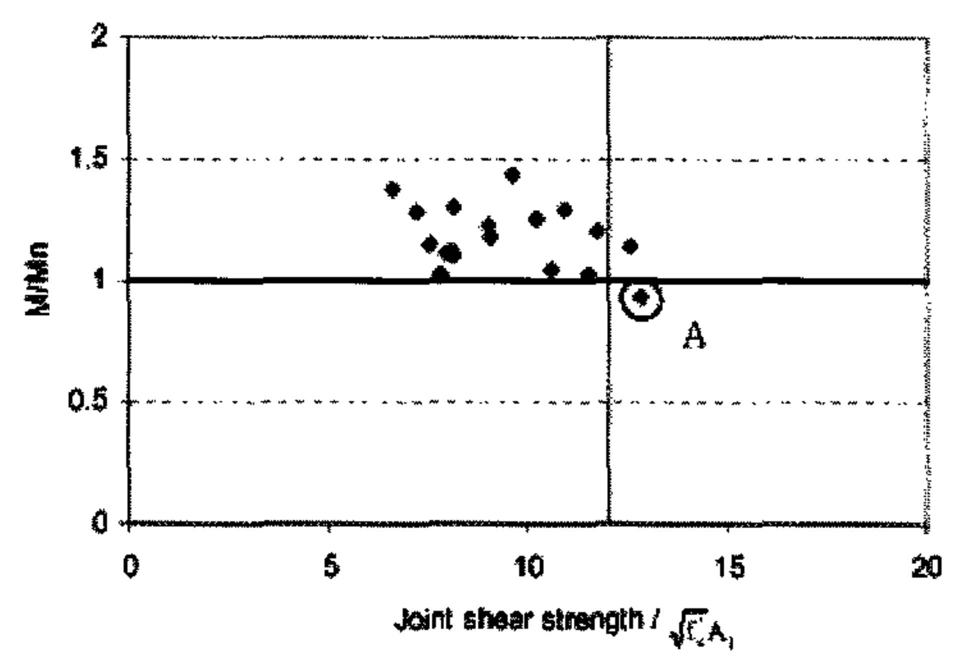
No.	Researcher	Year	Description	No. of specimens
1	Hanson, N.W. and Connor, H.W.	1967	ext. beam-col conn. (2-D frame)	5
2	Hanson, N.W.	1971	int. or ext. beam-col connections w/ or w/o trans. beam	5
3	Uzumeri, S. M. and Seckin, M	1974	ext. or isolated beam-column joint	5
4	Fenwick, R.C. and Irvine, H.M	1977	int. beam-col conn. (2-D frame)	4
5	Meinheit, D.F. and Jirsa, J.O.	1977	int. beam-col conn. (2-D frame)	14
6	Lee, D.L.N., Wight, J.K. and Hanson, R	1977	ext. beam-col conn. (2-D frame)	3
7	Soleimani, D., Popov, E.P. and Bertero, V.V.	1979	int. beam-col conn. (2-D frame)	2
8	Forzani, B, Popov, E.P. and Bertero, V.V.	1979	int. beam-col conn. (2-D frame)	2
9	Viwathanatepa, S., Popov, E.P. and Bertero, V.V.	1979	int. beam-col conn. (2-D frame)	2
10	Beckingsale, C.W., Park, R., and Paulay, T.	1980	int. beam-col conn. (2-D frame)	3
11	Scarpas, A.	1981	ext. beam-col conn. (2-D frame)	3
12	Milburn, J. R. and Park, R	1982	int. & ext. beam-col conn. (2-D frame)	4
13	Ehsani, M.R. and Wight, J.K.	1982	ext. beam-col conn. w/ or w/o trans. beams & slab	12
14	Durrani, A.J. and Wight, J.K.	1982	int. beam-col connections w/ trans. beams & slab	6
15	Joglekar, M.,Murry, P., Jirsa, J. and Kllngner, R.	1984	int. beam-col conn. w/ transv. beams and slab	4
16	Durrani, A.J. and Zerbe, H.E.	1987	ext. conn with transv. beam and/ or slab	2
17	Ehsani, M.R.,Moussa, A.E., and Vallenilla, C.R.	1987	ext. beam-col conn. (2-D frame), f'c=9380psi	5
18	Ruitong & Park, R.	1987	int. beam-col conn.	4
19	Alameddine, F. and Ehsani, M.	1991	ext. beam-col conn. (2-D frame), f'c=11000psi	3
20	Cheung, P.C., Paulay, T.,and Park, R.	1991	int. beam-column-slab joint of a one- or two-way frame	3
21	Raffaelle, G.S. and Wight, J.K.	1995	ext. beam-col conn to account for eccentricity	4
22	Chen, C-C. and Chen, G-K.	1999	ext. beam-column conn. (corner joint)	6
23	Quintero, C.Q. and Wight, J.K.	2001	int. wide beam-col connections w/ trans. beams & slab	3
24	LaFave, J. M. and Wight, J. K.	2001	int. wide or conventional beam-col connections w/ trans. beams & slab	2
25	Teng. S. and Zhou, H.	2003	int. beam-col conn. to account for eccentricity	6

only a single beam or under monotonic loading. To account for the nonlinearity of the elastic response, the procedure recommended in the ATC 342[8] was used to define the values at yield.

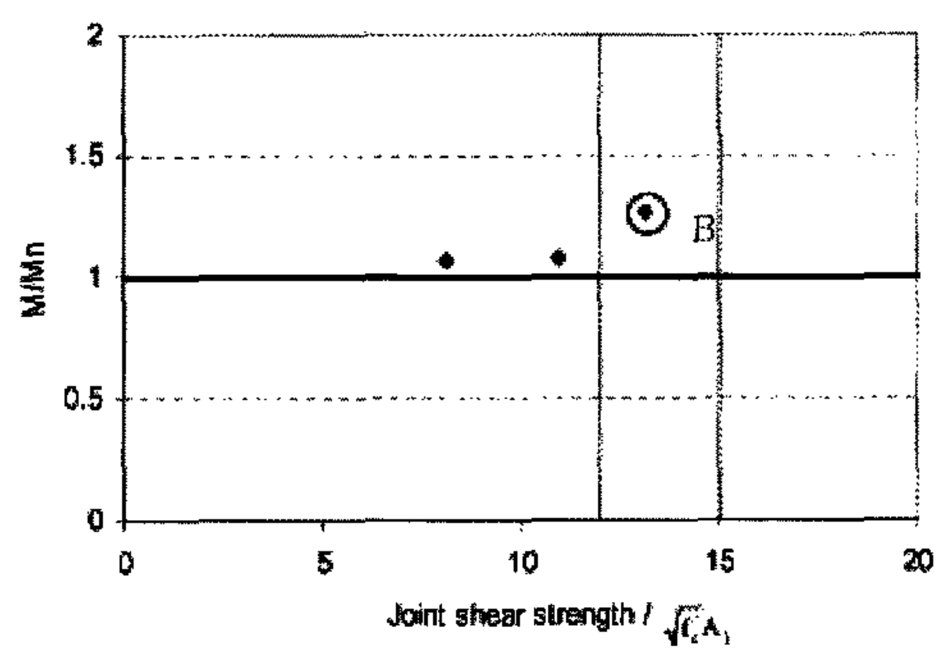
Figs. 2 through 4 show the test results for the SMF structures conforming to the requirements of Chapter 21 of the ACI 318-02. The data were analyzed and plotted based on different design parameters. M/M_n was calculated by dividing the measured moment strength from the test (M) by the nominal moment strength (M_n). The nominal moment strength was calculated in accordance with the ACI 318 requirements using the specified yield strength of the reinforcement and the specified compressive strength of the concrete. This M_n is analogous to M_p of the AISC Seismic Provisions. The measured moment strength corresponds to a plastic rotation of 3%. According to the AISC Seismic Provisions, a 3% plastic rotation is the minimum rotation capacity that a SMF connection should be able to reach without any major strength degradation. Thus, the ratio of M/M_n that is larger than 1.0 means that the measured moment strength at 3% plastic rotation is larger than the nominal strength of the beam. In such cases, the test specimens satisfies the minimum acceptance criteria based on the AISC Seismic Provisions. A total of 36 specimens fall within the SMF category. Out of these 38 specimens, 37 test specimens have a M/M_n ratio larger than 1.0, which means that they satisfy the acceptance criteria of the AISC Seismic Provisions.

2.1 Effect of Joint Shear Strength

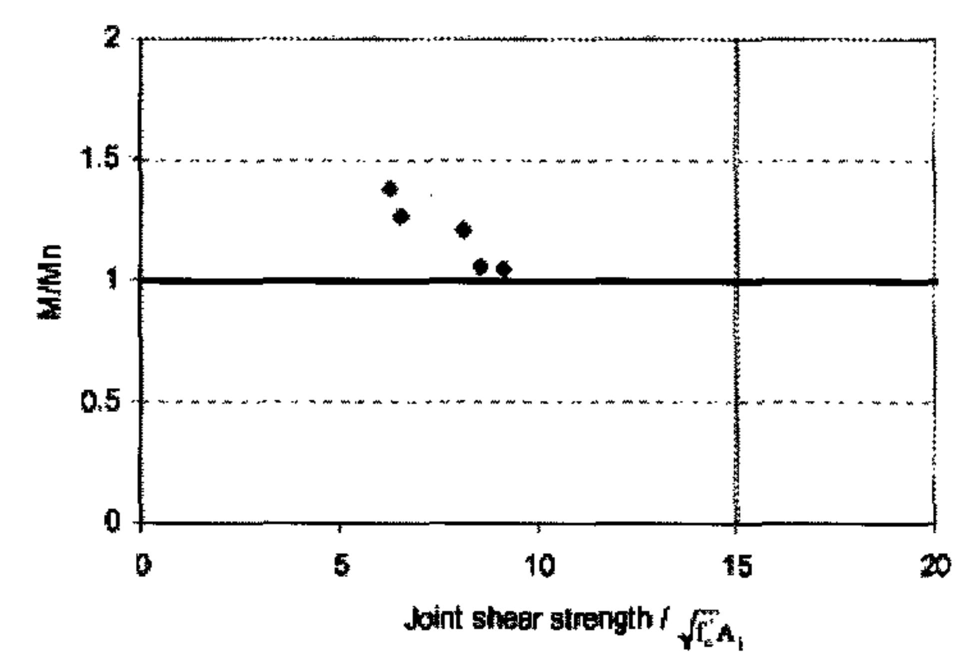
The current seismic design approach for beam- column joints in the United States, as specified in the ACI 318 Chapter 21, has mainly focused on providing adequate confinement to resist shear stresses in the joint. Past experimental studies have shown that the horizontal cross-sectional area and the degree of confinement of the joint are major variables affecting the shear strength of beam-column joints. The ACI 318-02 provisions define the shear strength of a joint as a function of



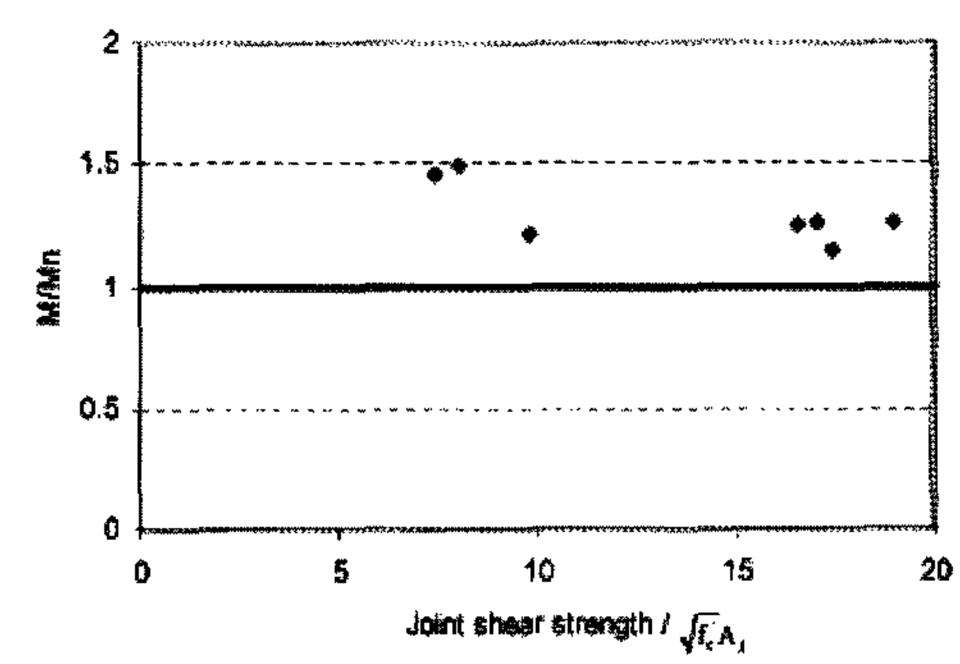
(a) Exterior connections without transverse beam



(b) Exterior connections with transverse beam



(c) Interior connections without transverse beam



(d) Interior connections with transverse beams

〈Fig. 2〉 Performance criteria as a function of joint shear stress for SMF connections the compressive strength of concrete, f_c', and the effective area of the joint, Aj, defined in Section 21.0. Allowable joint shear strength has three values based on the geometric configuration of a beam-column joint: (1) for joints confined on all four faces-$20\sqrt{f_c'}A_j$, (2) for joints confined on three faces or on two opposite faces-$15\sqrt{f_c'}A_j$, and (3) for all others-$12\sqrt{f_c'}A_j$. A confined column face is one that has a beam framing in, with the width of the beam being no less than three-quarters of the width of the column face.

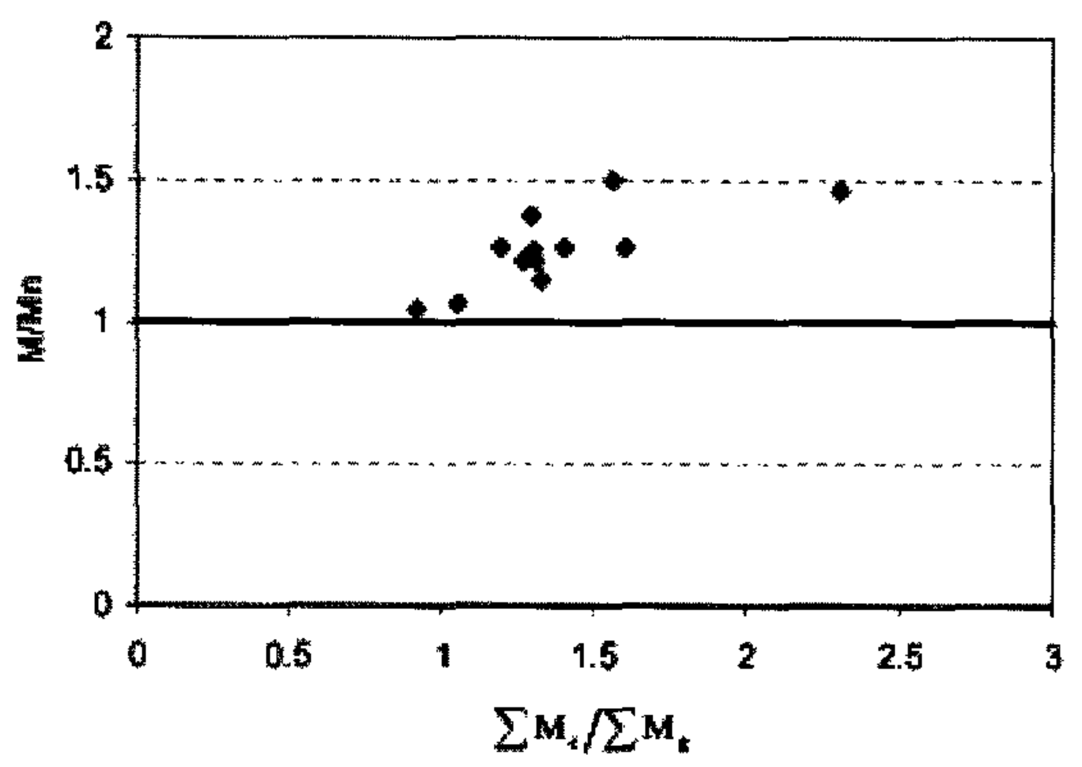
Fig. 2 shows the M/M_n ratios as a function of joint shear stress for the 38 specimens. Based on the confinement conditions, which account for the geometric configuration of the joints, four different graphs were created to evaluate the effect of joint shear stress. The maximum limit on the joint shear stress of the ACI 318-02 provisions are also shown in the figures, based on the joint configuration. For example, a line was plotted at a value of 12 where the maximum joint shear stress allowed by the ACI provisions is $12\sqrt{f_c'}$. Two limitations are drawn in Fig. 2 (b) corresponding to two confined faces that are perpendicular to each other, or three confined faces.

Comparisons of the figures clearly show that the performance of beam-column joints deteriorates as the joint shear stress increases. Figs. 2 (a) and (b) illustrate the performance of exterior beam-column joints without and with transverse beams, respectively. These correspond to cases where the permissible joint shear stress is 12 or $15\sqrt{f_c'}$. For the interior joints, the effect of the presence of transverse beams on the behavior of a beam-column joint can be observed by comparing Figs. 2 (c) and (d). As shown in the figures, the joints with transverse beams exhibit better performance, since transverse beams provide additional confinement to the joint. Figs. 2 (b) and (d), which exemplify the effect of transverse beams, show that the test specimens exhibit good performance even though the joint shear stresses are close to the limit specified in the ACI 318-02.

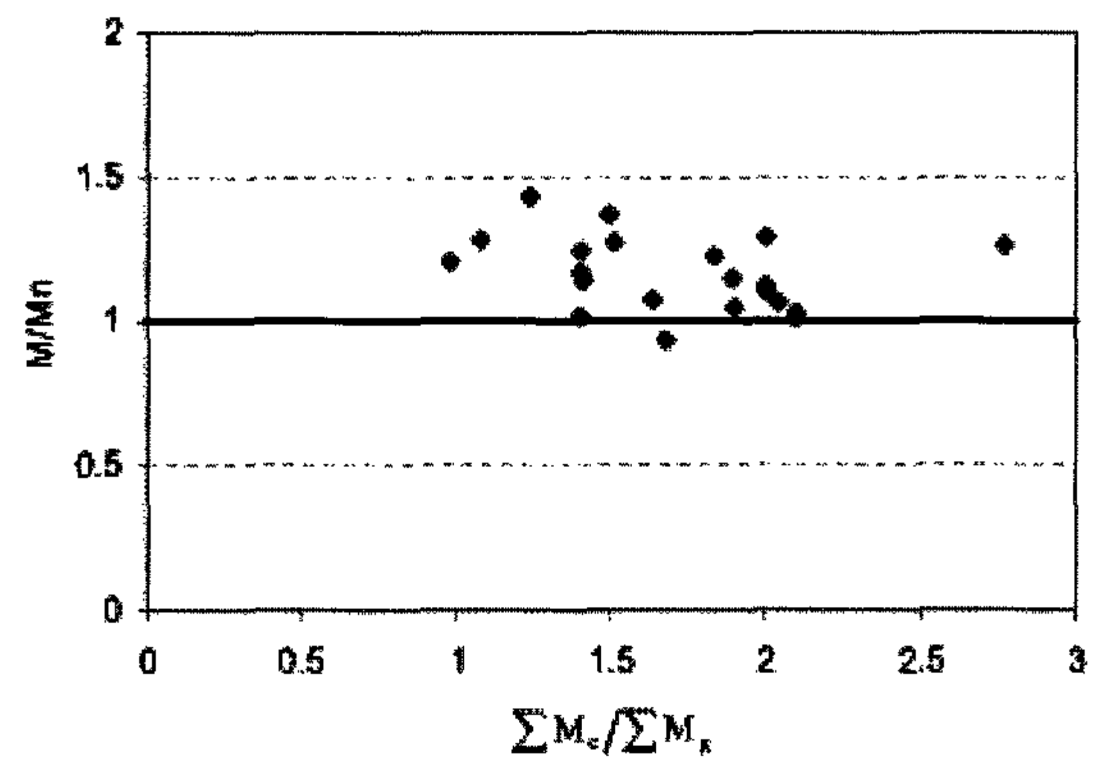
Two specimens-the one that did not satisfy the minimum performance criteria and the other that deviated from the major trend of diminished performance with increase in joint shear stress were marked as A and B and were examined in detail. Specimen A is one of the five test specimens subjected to a high level of joint shear stresses. Even though the column-to-beam flexural strength ratio for this specimen was 1.7, the high joint shear stress resulted in a shear failure of the joint rather than flexural hinges forming at the end of the beam framing into the joint. This undesirable failure mode was accompanied by a significant reduction in the stiffness and the strength of the specimen. Specimen B was tested to evaluate the effect of transverse beams. Two identical specimens were built; and the only difference between them was the existence of transverse beams in one of the specimens. The specimen with the transverse beams showed higher strength as well as higher initial and post-yielding stiffness compared to the specimen without transverse beams. As noted above, transverse beams provided additional confinement of the joint and increased shear resistance. As a result, the specimen with transverse beams was able to maintain adequate strength and stiffness during the test.

2.2 Effect of Column-to-Beam Flexural Strength Ratio

Fig. 3 (a) and (b) show the performance of the beam-column joints as a function of flexural strength ratios. Two figures were created to differentiate between interior and exterior joint configurations. Since all calculations were based on the actual measured strengths of the concrete and the reinforcement, the flexural strength ratios of some specimens are less than 1.2, which is the minimum requirement specified in Section 21.4.2.2 of the ACI 318-02. Note that the flexural strengths of the beams



(a) Interior joint connections



(b) Exterior joint connections

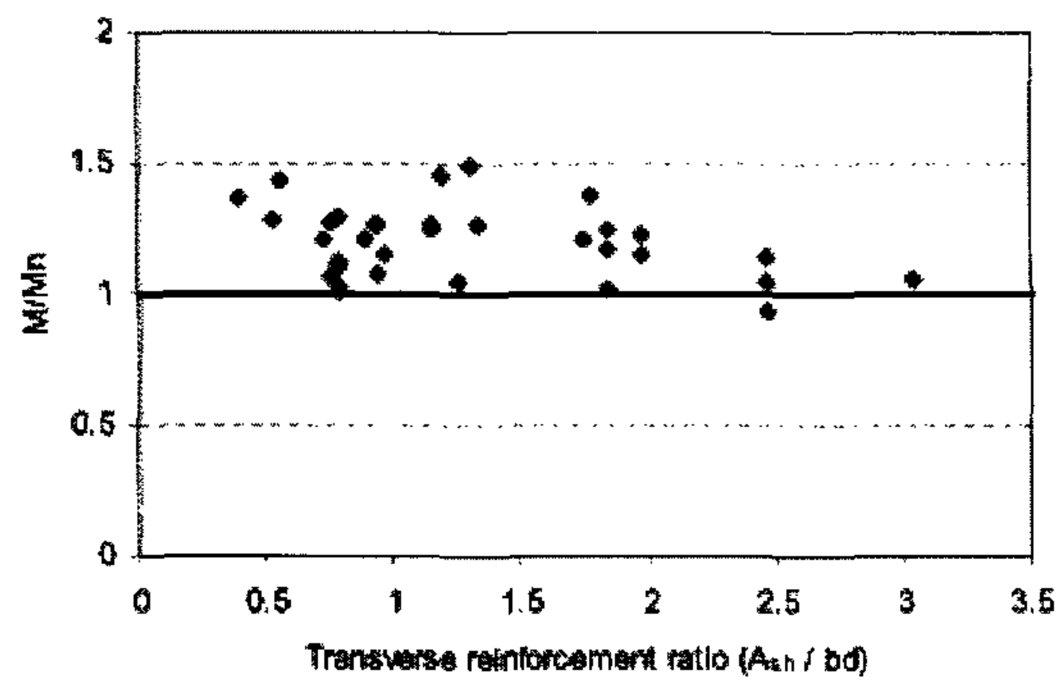
〈Fig. 3〉 Performance criteria as a function of joint flexural strength ratio of column to beam for SMF connections

included the contribution of the slab in compression for positive moment and of the slab reinforcement for negative moment, using the effective flange width specified in Section 8.10 of the ACI 318-02. All interior connections satisfied the AISC Seismic requirements. As the flexural strength ratio increases, M/M_n ratios also increase, which means that up to a 3% plastic rotation, the joint is able to maintain higher flexural strength than the nominal flexural strength of the beam. For the exterior joints, it is interesting to note that the flexural strength ratio and the performance of the connection are not directly correlated. This is primarily due to the different geometric configurations. Unlike an interior joint that has two columns and two beams framing into the joint in one plane, an exterior joint consists of two columns and one beam. Thus, the column-to-beam flexural strength ratio requirement, which is based on the sums of column and beam flexural strengths, can be easily met at exterior joints. Satisfying the column-to- beam flexural strength ratio requirement at interior joints provides additional column strength and better confinement of the joint.

2.3 Other Factors Affecting Performance

Other factors affecting the performance of beam-column joints are the transverse reinforcement ratio, the beam depth ratio, and the column axial load. Fig. 4 shows the performance of the joints as a function of the transverse reinforcement ratio, which is the area of transverse reinforcement in the joint divided by the effective area within the joint. The effective area was calculated by multiplying the column width and the beam effective depth. It is clear that the amount of transverse reinforcement in the joint is one of the key factors in achieving adequate strength and ductility, especially for large cycles of inelastic deformations. However, Fig. 4 does not clearly show that a larger transverse reinforcement ratio produces better performance. One of the main reasons is that more than half of the specimens tested prior to the early 1990's were built using less than full-scale models. To conform to the design requirements specified for SMF joints, the final designs turned out to have relatively small beam and column dimensions with large reinforcement ratios. As a result, some specimens have relatively large transverse reinforcement ratios.

The effects of the beam depth ratio and the column axial force are not shown here, because these effects are considered to be relatively insignificant. However, many researchers pointed out that column axial loads up to a certain range provide beneficial effects by improving bond conditions in the joint area and increasing the shear strength of the joint. In addition, the effect



〈Fig. 4〉 Performance criteria as a function of transverse reinforcement ratio for SMF connections

of bond stresses on the beam reinforcements were not examined in detail. Because high bond stresses in the joint region are believed to result in deterioration of joint shear strength, the ACI 318-02 provisions place a limit of 20 on the ratioof column depth-to-beam reinforcement diameter. All the test specimens have to conform these requirements to be classified as SMF connections.

In general, the SMF joints exhibited turned out to show satisfactory performances. 37 out of the 38 test specimens provided sufficient strength and ductility up to the plastic rotation level of 3%. The factors such as joint shear stress, column-to-beam flexural strength ratio, and transverse reinforcement ratio in a joint all played a key role in producing satisfactory performance of the joints.

3. Conclusions

This study examined and compared the experimental results on RC beam-column joint connections for inelastic cyclic deformation capacity. The following conclusions were drawn from this study.

1. Most of the joints that satisfied the design requirements for SMF structures of the ACI 318-02 showed high M/M_n ratios, which indicates that these connections are ductile up to a plastic rotation of 3% without any major degradation in strength. This is mainly due to the stringent ACI 318-02 requirements for SMF joints.
2. The presence of transverse beams increases confinement and shear resistance of joints, which results in better performance than for joints without transverse beams. All of the SMF connections that satisfy the ACI 318-02 limitations on joint shear stress turned out to meet the acceptance criteria.
3. As expected, a high column-to-beam flexural strength ratio increases the performance of interior joints. The same effect, however, was not observed in exterior joints. It is believed that the configuration of the interior joint (with 2 columns and 2 beams) versus that of the exterior joint (2 columns and 1 beam) offers the necessary explanation.

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