A STUDY OF SHEAR BEHAVIOR OF REINFORCED CONCRETE BEAM-COLUMN JOINTS

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A B S T R A C T

The need for adequate joint shear capacity in reinforced concrete frame structures has been recognized by many design and research engineers. Shear design approaches to beam-column joints have been based primarily on beam shear mechanisms or truss mechanisms. A compression strut mechanism is presented in this report to evaluate shear strength according to observations from laboratory tests and in the field. The strength of the inclined strut is a function of the concrete strength, column axial load, joint geometry, transverse reinforcement, lateral beams, and cyclic loading. The simple equations established in this report represent quite accurately the observed results from 164 test specimens from the U.S., Japan, New Zealand, Canada, U.S.S.R., and China. Design recommendations are presented for joint shear strength and design examples are presented using the proposed design method.

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CHAPTER 1

INTRODUCTION

1.1 Background

The ability of many multistory reinforced concrete frames to resist strong earthquakes has been demonstrated in Chile 1960 [1], Yugoslavia 1963 [2], Alaska 1964 [3], Caracas 1967 [4,5], Tokachi-Oki 1968 [6], and San Fernando 1971 [7]. However, excessive damage to other structures has verified the need for recommendations for design of frame structures, particularly with respect to detailing of reinforcement. Inadequate performance of some structures could be attribued to beam-column joints.

In designing a building to withstand severe earthquakes, it is necessary that seismic energy be absorbed and dissipated through large but controllable inelastic deformations of the structure. The sources of potential brittle failure must be eliminated. Thus, it is necessary to prevent premature crushing and shearing of concrete as well as sudden loss of bond and anchorage. To utilize the energy-dissipating capacity of structural members, the joint connecting the beams and columns must function without brittle failure taking place and without excessive loss of stiffness.

The poor performance of some frame structures in severe earthquakes has stimulated researchers to study behavior of frame components. One area receiving considerable attention is the connection detail. Tests have shown that seismic loading can induce yielding of the joint ties followed by loss of integrity of the joint and substantial degradation of strength and stiffness. Such results have been observed in both interior and exterior joints.

About 300 joint tests have been reported in the literature. This large volume of data is summarized in this report and is used to develop a rational procedure for joint design. The data used are based on tests from the U.S., Japan, New Zealand, Canada, U.S.S.R., and China.

1.2 Objectives

The purpose of this report is to develop an approach to determine the shear strength and behavior of beam-column joints under monotonic and cyclic loading. The specific objectives of the report are as follows:

(1) To develop a rational approach to joint shear strength adopting a compression strut mechanism. At present, design approaches to beam-column joints have been based on either beam shear mechanisms or truss mechanisms. The two methods are generally conservative.

(2) To examine a variety of factors affecting joint shear strength: concrete strength, geometric parameters, column load, transverse reinforcement, lateral beams, and cyclic loading.

(3) To assess the effectiveness of current design recommendations for ensuring satisfactory shear behavior of joints.

CHAPTER 2

REVIEW OF RESEARCH AND PRACTICE

2.1 A Review of Previous Tests

A wide variety of tests on beam-column joints of reinforced concrete frames have been undertaken since 1967 [8-48]. Because of space limitations no attempt will be made to describe all tests in detail. However, the intent is to give a general review of the subject by describing briefly some selected programs. Data from most of the test programs are presented in tabular form in Appendix A.

Hanson and Conner [8-10] conducted some of the earliest studies of the behavior of beam-column assemblies in the U.S. They concluded that subassemblies containing joints must behave in a ductile manner if frame structures are to perform satisfactorily in a major earthquake. From sixteen tests, they concluded that (1) it is necessary to provide transverse reinforcement to resist joint shear; (2) a frame reinforced with Grade 60 steel has adequate ductility to provide a total reserve energy capacity necessary to prevent collapse during a severe earthquake; and (3) adequate energy absorption for seismic resistance can be provided near the junction of beam and column if proper attention is given to anchorage, shear resistance, and confinement.

The design approach suggested by the authors was derived directly from equations for beam shear, perhaps because of the lack of a suitable structural model for the joint. Beam hinges generally occurred in early cycles in the tests. As loading progressed, either bond failure or shear failure in the joint panel was observed.

Meinheit and Jirsa [11] carried out a series of tests at the University of Texas. The purpose was to define the shear strength of joints and derive minimum requirements for the joint to maintain integrity under given loading conditions. The variables considered include: column load, joint hoop reinforcement, lateral beams, concrete strength, the size of the column, and the beam and type of load. In most cases the beams did not yield. A proposed design equation reflected the important variables from the tests and provided a better understanding of the shear behavior of the beam-column joint.

Soleimani, Popov, and Bertero [13] reported hysteretic behavior of reinforced concrete beam-column subassemblages with the emphasis on the influence of anchorage capacity of bars in joints. They drew the following conclusions: (1) after a loss of bond of the main beam bars in a joint, the inelastic deformations are concentrated at the beam fixed ends. When the bond of the beam bars in a joint is lost, the stability of the whole structural system may be jeopardized; (2) even in situations where a complete loss of bond of the main beam bars does not occur during cyclic reversals, the beam fixed-end rotations remain important because the concentrations of inelastic deformations at the fixed ends can contribute between 20 to 35% to the total lateral displacement of a subassemblage.

Park and Paulay [19] have reported the results of tests conducted in New Zealand on thirteen full-scale reinforced concrete beam-column assemblies. The major variables tested were the amount and arrangement of transverse steel in the joint and the method of anchoring the beam steel. The principal conclusions from these tests are: (1) the critical joint crack forms along the joint diagonal rather than at a 45° angle, and (2) the contribution of joint concrete to shear resistance should be neglected and only two-thirds of the shear reinforcement should be considered to be at yield or steel must increase by 50%. Subsequent tests by Paulay and Scarpas [77] showed that the transverse reinforcement requirements implied by the tests reported in Ref. 19 could be greatly reduced if the joint was reinforced by intermediate column bars.

Paulay, Park, and Priestley [22] point out that the shear force applied to a joint core should be apportioned between the concrete diagonal strut which exists between the compressed corners of the joint core and the truss mechanism consisting of horizontal and vertical stirrup ties and longitudinal bars. The shear resistance of the concrete in the joint is primarily due to the contribution from the diagonal strut, but this diminishes when plastic hinges form in the beams adjacent to the column faces. The shear resistance provided by reinforcement is due to the contribution from truss action.

Many reports [25-42] on beam-column joints have been published in Japan in recent years. The Japanese research covers a wide range of topics, such as hysteretic behavior and failure characteristics of joints, effects and types of joint transverse steel and lateral beams, and anchorage of beam reinforcement in joints.

Hamada and Kamimura [25,26] report tests which indicate that (1) beam plastic hinges developed at the faces adjacent to the joint, crushed concrete was observed; (2) an inclined strut formed in the joint and the resultant compression carried by the strut increased with loading until crushing of the strut occurred; and (3) bond failure between the concrete in the joint zone and the beam flexural reinforcement occurred in several tests. However, this did not decrease the shear resistance of the joint, nor did it influence the strain in the joint ties. The tests demonstrated the effectiveness of the diagonal concrete strut in resisting joint zone shear.

Ohwada [27-29] investigated the effect of lateral beams on joint ultimate strength. Three specimens had no lateral beams, three had lateral beams on one side and three had lateral beams on two sides. The report indicated that joints with no lateral beams had the lowest capacity. With lateral beams on one side of the joint, the capacity was improved only slightly. The most improvement was noted with beams on both sides. Concrete along the diagonal of the specimens with no beams or on one side only exhibited crushing, but this was not observed in the case of lateral beams on both sides. The lateral beams influenced the hysteretic behavior and deterioration of concrete in the connections. The lateral beams appeared to improve the hysteretic behavior by confining the concrete.

Uzumeri and Seckin [43-44] reported a series of seventeen tests conducted at the University of Toronto on exterior joints with varying levels of column axial load. Most specimens failed in the joint region. The report indicates that the function of joint stirrups is to provide confinement and shear resistance to the joint and to help maintain anchorage of beam steel through the joint over a number of reversals of load. In general, the performance of the joint was improved when transverse reinforcement in the joint did not yield. The report [44] also indicates that the magnitude of column axial compressive force had little effect on the behavior of a well-designed exterior joint.

Bychenkov, et al [45,46] at the Institute for Concrete and Reinforced Concrete (USSR) showed that compression failure in the central zone of the joint occurred along a diagonal direction. Nineteen interior joints were tested. The magnitude of compressive deformations in the joint concrete prior to fracture is close to the ultimate strength of concrete under concentric compression. The strength of joints under a skew-symmetrical load may be estimated as the strength of an inclined concrete prism. Failure of the joints under cyclic reversed skew-symmetrical loads was similar to that under monotonic skew-symmetrical loads; that is, the compressive strength of the concrete in the inclined direction was exhausted. The joint strength was substantially increased by providing welded mesh as transverse reinforcement in the central zone.

The Research Group on Anti-Seismic Joints in China [47] carried out a series of tests on exterior joints. Reinforced concrete frame structures with cast-in-place columns and precast beams are widely used in China. Preliminary conclusions obtained from the tests indicate that the earthquake-resistant behavior (stiffness, mode of failure) of cast-in-place column-precast beam joints was basically the same as monolithic joints. Therefore, cast-in-place column-precast beam joints could be treated as monolithic joints for design purposes.

Gavrilovic, et al [48] in Yugoslavia investigated the effect of plain reinforcement on joint behavior. The conclusion was that plain reinforcement did not perform adequately under cyclic loading in the domain of severe deformations. Deterioration of bond in the panel zone was observed.

Summary. The vast amount of research work done on subassemblage behavior to date can be summarized as follows. The factors that influence joint shear strength include column load, joint horizontal and vertical reinforcement, lateral beams, concrete strength, and type of load. Many reports have demonstrated the effect of concrete on resisting shear in joints. The lateral beams can contribute to the joint ultimate strength by confining the concrete. The function of joint shear reinforcement is to provide confinement and shear resistance to the joint. The influence of the level of column load on joint shear strength may be a factor. A diagonal compression strut may form in a joint under monotonic or cyclic loading. Therefore, the concrete contribution to the joint shear strength cannot be ignored. Where bond between concrete and beam bars in a joint fails, the resultant force in the strut will not be influenced by the loss of bond though the loss of bond may affect the stability of the whole structural system.

2.2 Actions on Plane Frame Joints

The forces acting on an interior joint are shown in Fig. 2.1. The horizontal shear force in this zone is generated by the flexural forces from the two beams and the shear in the column. If the axial

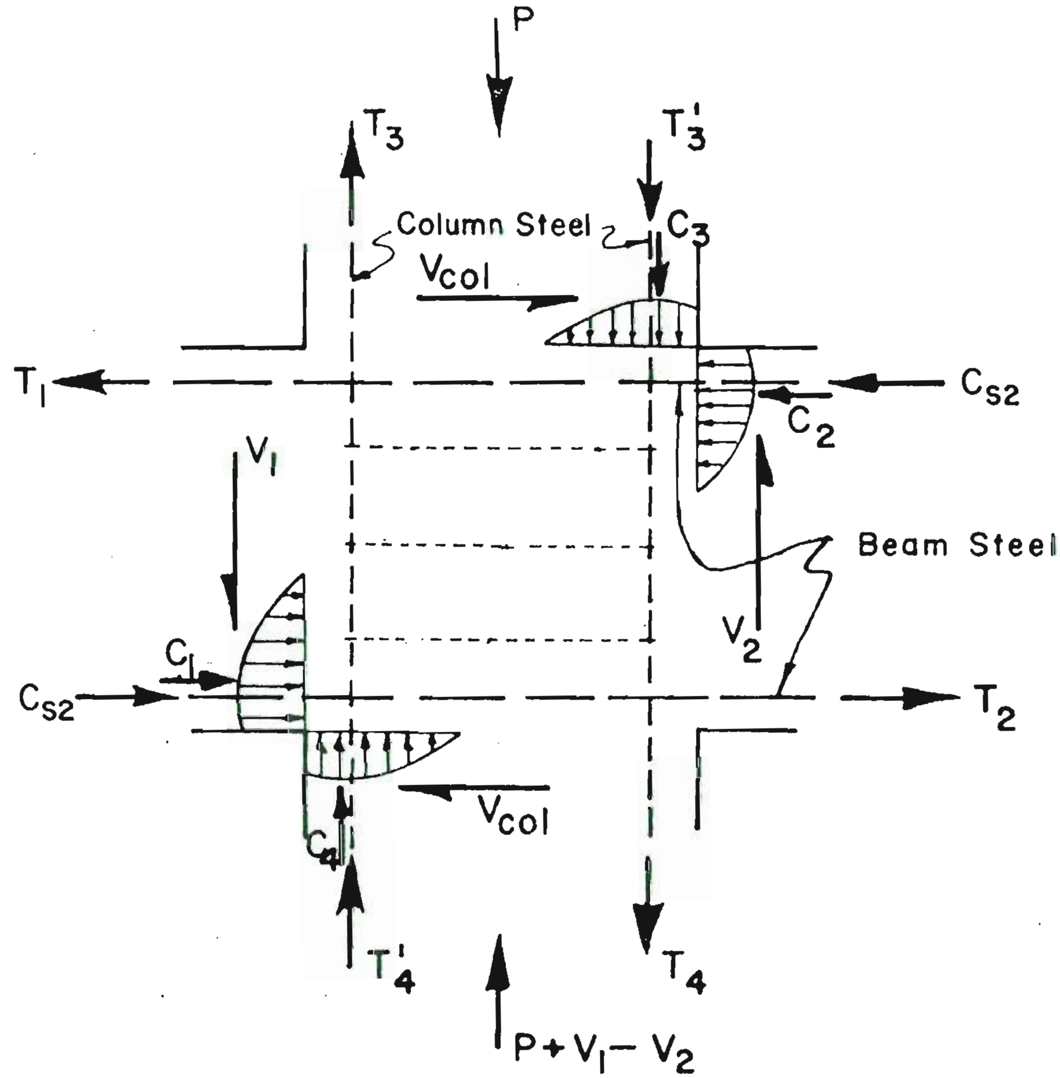


Fig. 2.1 Forces acting on a joint

load in the beams is negligible, the horizontal shear force is given by

$$V_j = T_1 + T_2 - V_{col} \qquad (2.1)$$

where T_1 and T_2 are the flexural tension forces in the beams and V_{col} is the shear force in the column.

In structures designed using the strong column and weak beam concept, plastic hinges may form in the beams. The hinges may limit the shear imposed on the joint. If large deformations are imposed on the structure, the flexural tension reinforcement may strain harden and it is necessary to allow for this possibility in determining the maximum action which can be imposed on the joint.

It should be noted that an equation similar to Eq. (2.1) may be derived for vertical shear force acting on the joint.

The forces imposed on a plane frame exterior joint are similar to those acting on an interior joint. In this case the horizontal joint shear is given by:

$$V_j = T_1 - V_{col} \qquad (2.2)$$

2.3 Shear Resistance Mechanisms and Shear Design Methods

Since the 1960s, many investigations of beam-column connections have been reported in the literature. Test results have been interpreted in several different ways as investigators have applied different, and in many cases ill-defined, performance criteria. The different mechanisms include beam shear, joint truss (panel truss), and joint compression strut mechanisms. As different criteria have been applied, different design techniques have arisen which reflect the assumptions made.

Three different methods which have been translated into design specification format are available. These methods are included in the ACI-ASCE Committee 352 Report [51]. The Draft

New Zealand Standard [52], and the Tentative Provisions of the Applied Technology Council [53].

Beam Shear Mechanism and ACI-ASCE 352 Method. For determining joint shear strength, ACI-ASCE Committee 352 [51] adopted the approach used for beam shear. The shear strength contributed by the shear reinforcement may be developed from the truss analogy. Assume that a diagonal crack (at about 45°) extends through the joint. Shear reinforcement crosses the inclined crack as in a beam (Fig. 2.2). The shear strength contributed by the concrete is taken equal to the shearing force causing diagonal cracking. The shear capacity is the sum of the contribution from the transverse steel and the concrete. Though the procedure adopted by ACI-ASCE 352 is identical to that for shear in beams, the equations have been modified to reflect data from beam-column tests.

The designer may work with the joint shear (V_j) (Fig. 2.2a) or with joint shear stress

$$v_j = V_j/bd = v_s + v_c \tag{2.3}$$

where b = width of the compression face of the column

d = distance from extreme compressive fiber to the centroid of the tension force

The shear stress assigned to the concrete is

$$v_c = 3.5\beta\gamma \sqrt{f'_c(1 + 0.002\, N_u/A_g)} \tag{2.4}$$

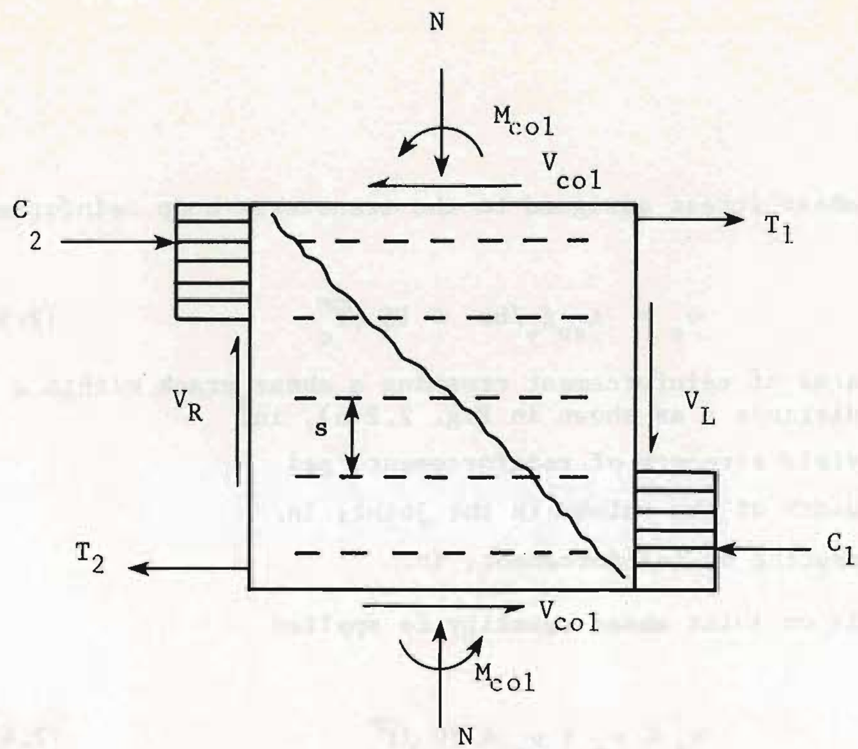
where β = factor reflecting severity of loading 1.4 for Type 1 joint; 1.0 for Type 2 (seismic) joint

γ = factor reflecting confinement by lateral beams - 1.4 if the confining members cover at least 3/4 of the width and 3/4 of the depth of both joint faces; 1.0 if the confining members do not meet the above requirements

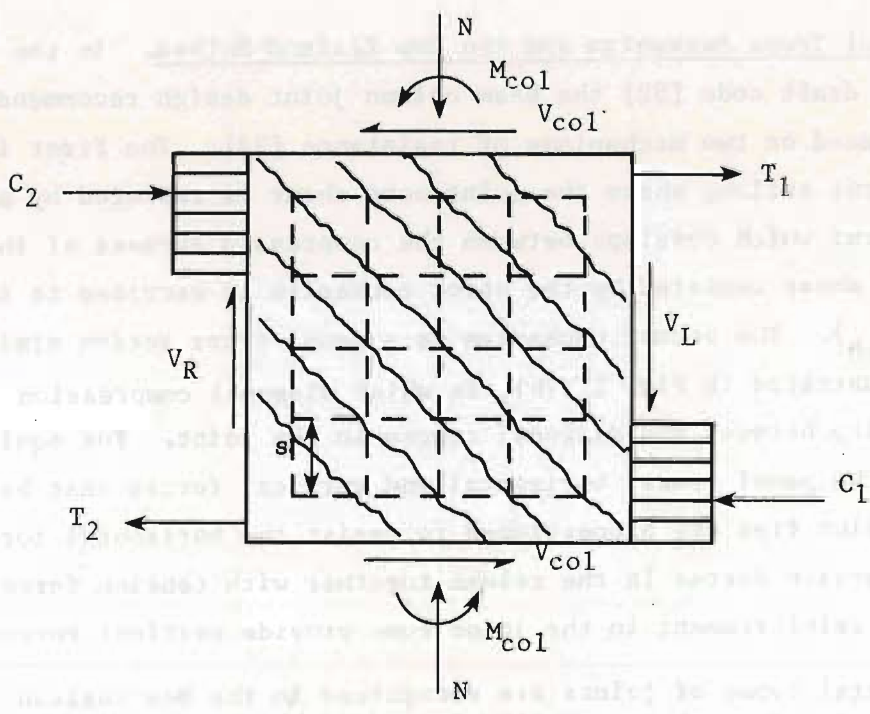
f'_c = concrete compressive strength, psi

N_u = magnitude of column load, lbs (if N_u is in tension, v_c is taken as zero)

A_g = gross area of column, in.^{2}



(a) Beam shear mechanism



(b) Truss mechanism

Fig. 2.2 Mechanisms of joint shear resistance

The shear stress assigned to the transverse hoop reinforcement is:

$$v_s = A_{sv} f_y / bs \leq 15 \sqrt{f'_c} \qquad (2.5)$$

where A_{sv} = area of reinforcement crossing a shear crack within a distance s as shown in Fig. 2.2(a), in.^{2}

f_y = yield strength of reinforcement, psi

b = width of the column in the joint, in.

s = spacing of reinforcement, in.

An upper limit on joint shear capacity is applied.

$$v_j \leq v_c + v_s \leq 20 \sqrt{f'_c} \qquad (2.6)$$

Panel Truss Mechanism and the New Zealand Method. In the New Zealand draft code [52] the beam column joint design recommendations are based on two mechanisms of resistance [22]. The first is diagonal strut action, where the joint zone shear is resisted by a diagonal strut which develops between the compressed corners of the joint. The shear resisted by the strut mechanism is ascribed to the concrete (V_{ch}). The second mechanism is a panel truss action similar to that illustrated in Fig. 2.2(b), in which diagonal compression forces develop between the diagonal cracks in the joint. For equilibrium of the panel truss, horizontal and vertical forces must be applied. Joint ties are proportioned to resist the horizontal forces, while compression forces in the column together with tension forces in vertical reinforcement in the joint zone provide vertical forces.

Several types of joints are recognized in the New Zealand Code. Where a joint is designed to remain elastic (or if it is not subjected to seismic action), the concrete is assumed to resist a considerable portion of the joint shear. Joint zone ties and vertical reinforcement are proportioned to resist the remainder. However, where inelastic cyclic loading occurs in the beams close to the column faces, the shear resisted by the concrete is greatly reduced. It is considered that the stiffness and strength degradation associated with diagonal strut action are unacceptable.

The design approach for a beam column joint where the beams are designed to hinge against the column faces under seismic conditions is outlined below.

The horizontal joint zone shear V_{jh} is determined by considering the forces acting on the joint (see Fig. 2.1):

$$V_{jh} = T_1 + T_2 - V_{col} \tag{2.1a}$$

where T_1 and T_2 are the tension forces in the beams intersecting at the joint calculated assuming that plastic hinges have developed in the beams. Allowance is made for the likely overstrength of the reinforcement and for strain hardening of this steel (i.e., $f_s = 1.25f_y$ for f_y = 275 MPa [≈ 40 ksi] reinforcement).

To prevent premature degradation of the joint due to compression failure of the concrete, the joint zone dimensions must be proportioned to satisfy the requirement

$$V_{jh} \leq 1.5\sqrt{f'_c}\, b_j h_c \quad \text{(2.7) (in MPa units)}$$

$$12\sqrt{f'_c}\, b_j h_c \quad \text{(in customary units)}$$

where h_c is the column depth and b_j is the joint width (plan dimension).

The horizontal joint zone ties are designed to resist a shear force of V_{sh} where

$$V_{sh} = V_{jh} - V_{ch}$$

The value of the shear resisted by the concrete V_{ch} varies with the level of axial load P_e on the column as indicated below.

$$\text{For } P_e \leq 0.1f'_c A_g/C_j \tag{2.8}$$

$$V_{ch} = 0.0$$

where A_g is the gross area of the column and C_j is 1.0 for planar frame joints and less for joints subjected to biaxial forces in space frames.

Where P_e exceeds the limit in Eq. (2.8), V_{ch} is given by:

$$V_{ch} = \frac{2}{3}\left(\sqrt{\frac{C_j P_e}{A_g} - \frac{f_c'}{10}}\right) b_j h_c \tag{2.9}$$

The total area of joint zone ties A_{jh} required in the joint is given by the expression

$$A_{jh} = V_{sh}/f_y \tag{2.10}$$

where f_y is the yield stress of these ties.

The forces acting on the joint in the vertical direction are examined to determine the value of the shear force V_{jv}. If the columns have been designed to prevent the formation of column hinge, the shear resisted by the concrete V_{cv} is given by

$$V_{cv} = \frac{A_{sc}'}{A_{sc}} V_{jv} \left(0.6 + \frac{C_j P_e}{A_g f_c'}\right) b_j h_d$$

where A_{sc}' is the area of the compression reinforcement in one face of the column and A_{sc} is the area of tension reinforcement on the opposite face, and h_d is the overall depth of the beam. The value of V_{cv} is reduced if the column is subjected to axial tension.

Vertical joint zone reinforcement, which usually takes the form of intermediate column bars between the A_{sc}' and A_{sc} layers, is required to resist the shear given by

$$V_{sv} = V_{jv} - V_{cv}$$

and the area of vertical reinforcement is given by

$$V_{sv} = A_{sv} f_y$$

where f_y is the yield stress in the vertical reinforcement.

Applied Technology (ATC-3) Approach. To investigate the basic shear strength of beam-column joints where failure occurs before framing beams yield, Meinheit and Jirsa [11] carried out a series of tests which indicates that an ultimate shear stress of $20\sqrt{f'_c}$ can be considered for an unconfined joint in a nonseismic region. A reduction factor of 0.6 for a joint in a seismic zone was recommended. The influence of lateral beams is considered using a factor which is a function of the beam size. Transverse reinforcement in the joint can increase marginally the shear capacity. The concrete strength placed in the joint is also a very significant parameter in determining the ultimate joint shear strength. Therefore, the joint shear is a function of the three most significant variables: lateral beams, transverse joint hoop reinforcement, and concrete strength. A shear panel or a diagonal compression strut mechanism has been observed from the test. Only one equation was used to express the effects of the above factors with the emphasis on the concrete transferring shear forces in the joint. Transverse reinforcement is considered to confine the concrete and maintain its integrity. Confining reinforcement to meet ACI 352 minimum confinement provisions has been suggested.

The recommendations for joint shear in the "Tentative Provisions for the Development of Seismic Regulations for Buildings" [53] are based on the work done by Meinheit and Jirsa. For joints laterally confined on all four sides, the shear stress in the joint shall not exceed $16\sqrt{f'_c}$, that is:

$$v_u = \frac{V_u}{\phi bd} = \frac{V_u}{0.85bd} \leq 16\sqrt{f'_c}$$

For joints not confined, the maximum shear stress is reduced 25%. While no specific shear reinforcement is required to carry shear, minimum column confining reinforcement is required to be carried through the joint.

<u>Comparison of These Methods</u>. The contribution of the concrete to the joint shear strength recommended by ACI-ASCE Committee 352 originated from tests on members, not joints, subjected to compressive load in addition to shear and flexure. Since the shear span is quite different in the two cases, the results may not be

applicable to conventional joints in which the column depth is approximately equal to the beam depth [11, 49,50].

Since concrete is not taken into account in many cases using the Draft New Zealand Standard, a large number of transverse hoops are required in the joint. By checking 19 specimens available from the U.S., Japan, Canada, and New Zealand [8-11,23,31,33,43] which had volumetric percentage of joint transverse reinforcement $0 \le \rho_s \le 0.01$, a lateral beam on one side or no lateral beams and beam hinges at the face of the column, it was observed that the average shear carried by the joint under cyclic loading is about 66% of that under monotonic loading. The ratios of the value of shear strength at the worst peak under cyclic loading to the test value of shear strength at maximum peak after yielding under monotonic loading were distributed from 0.40 to 0.90, as shown in Fig. 2.3. This indicates that even under cyclic loading concrete in joints can still make a contribution to shear capacity. When joint reinforcement is increased and other conditions are kept constant as above, tests indicate that not all joint ties reach the yield stress. With inadequate vertical reinforcement in the joint zone, only two-thirds of the shear reinforcement could be considered to be at yield [16-19]. With well-detailed vertical reinforcement, all transverse steel is effective [77]. In fact, concrete in joints does sustain a certain amount of shear force, even though joint reinforcement may carry more shear. Accordingly, the shear resistance of the concrete as a diagonal strut in the joint core need not be ignored in design. The method in ATC-3 probably provides a lower bound on joint shear capacity. Although the concrete is assigned more shear, the column size must be increased because v_u is limited.

In short, the New Zealand method may represent the maximum reinforcement for the joint, ATC-3 method the minimum reinforcement, and ACI-ASCE method intermediate reinforcement. The differences arise from different objectives, as discussed before. The disparity between approaches indicates that no universally acceptable model has yet been

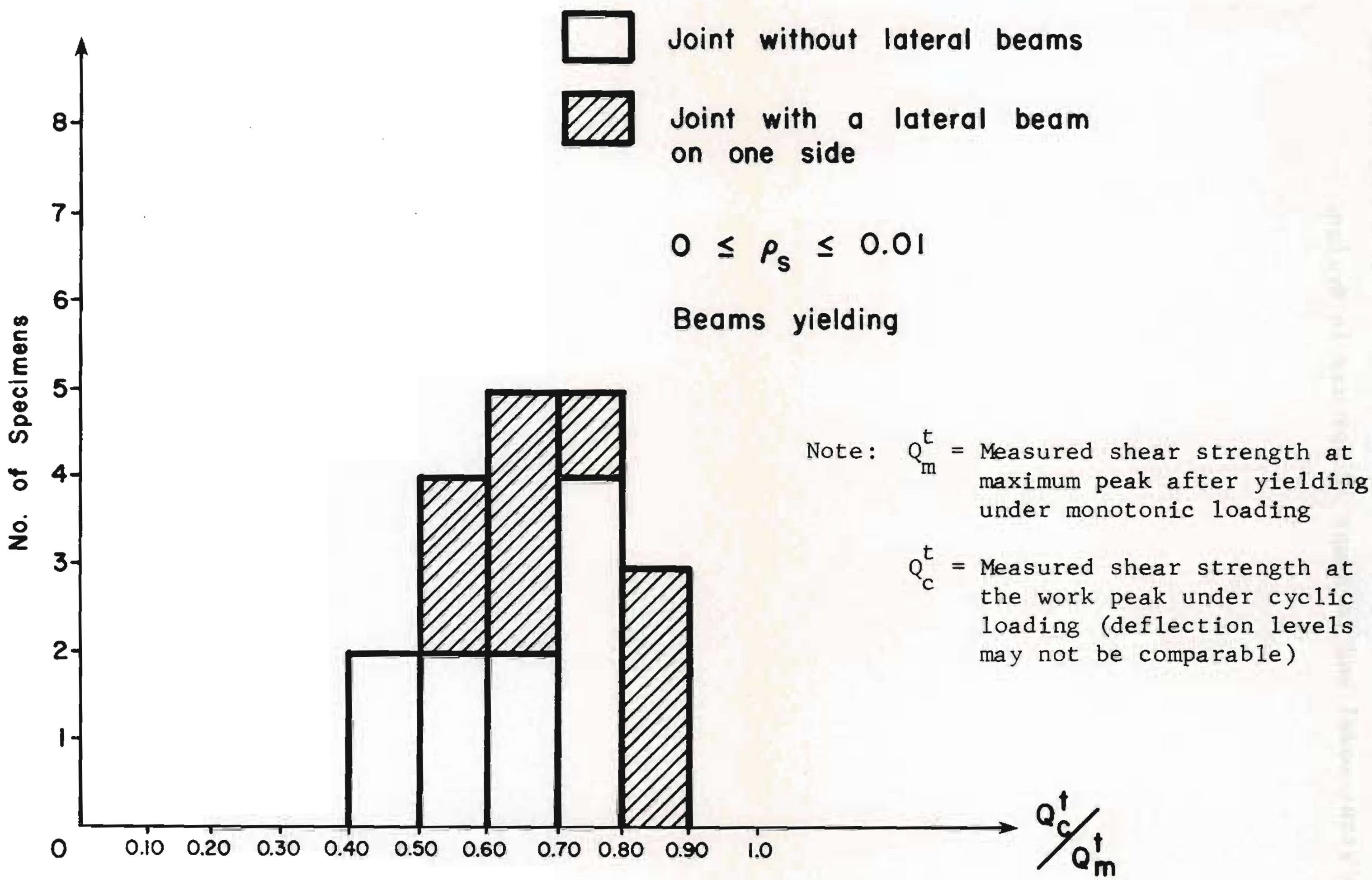


Fig. 2.3 Contribution of concrete to joint shear strength under cyclic loading

developed to simulate the behavior of the joint in shear. The purpose of this investigation is to expand the development of the compression strut model and to adapt it for use in design.

CHAPTER 3

STRUT MECHANISM OF JOINT SHEAR RESISTANCE

From a large number of test observations the modes of joint failure can be categorized as shear failure, anchorage failure, beam or column hinging failure, and compressive failure [49,50]. Sometimes it is hard to distinguish between shear failure and compression failure. If the shear forces transmitted from the beam and the column to the joint are large and little or no axial load is exerted on the column, shear failure may occur at the joint. If the shear forces are small and large axial load is exerted, compression failure may occur at the joint. It is often difficult to predict the exact failure mode of a specimen because of the influence of many factors.

3.1 Compression Failure of Joints

Researchers in Japan [25-30], the U.S.S.R. [45,46], and the U.S. [11] have conducted many tests on failure behavior of joints in recent years. Compression failure through the joint has been observed. The history and character of failure may be summarized as follows: (1) prior to forming cracks, the central zone of the joint is capable of carrying principal tensile and compressive stresses; (2) parallel inclined diagonal cracks form and extend with increase in loading; (3) after the formation of the cracks in the central zone of the joint, the concrete in the joint can transmit only compression; (4) the magnitude of compressive deformation in the joint concrete prior to failure reaches a value corresponding to the ultimate strength of concrete under concentric compression; (5) concrete in the central zone of the joint is crushed; (6) in the stage close to the ultimate strength of the joint, beam bars previously in compression are in tension at the face of the column in some specimens. Figures 3.1 to 3.3 show crack patterns at joints at failure. The patterns of cracking indicate that forces must be transferred through the joint by compression along the diagonal in the central portion of the joint--an inclined strut. This was checked experimentally in the U.S.S.R. by testing a specimen in which the central portion of the joint was made in the shape of a diagonal brace. The strength of joints under a skew-symmetric load may be estimated as the strength of an equivalent inclined concrete strut in which the above character of joint failure can be reflected. The strut mechanism appears to provide joint shear resistance from formation of first diagonal cracking to failure of the joint.

3.2 Inclined Strut

The compression zones in beams and columns provide an end condition favorable and necessary for the formation of a concrete strut. Figure 3.4 shows the distribution of principal compressive deformations in the central zone of a beam-column joint under skew-symmetric loading. The magnitude of the axial compressive strains along the diagonal direction (from upper left to lower right) at different sections is almost the same. The strain distribution on the cross sections shows that strains at the middle are larger and approach zero at the ends. The configuration of the strut is something like a shuttle (see Fig. 3.5).

As described above, the formation of a strut depends mainly on compressive forces transmitted from the beam and the column. Different magnitudes and distributions of compressive forces may result in different strut orientations, as shown in Fig. 3.6. Figure 3.6(a) shows a conventional joint where beam hinges occur at the face of the column and the column is stressed to less than ultimate at the top and the bottom of the joint. Figure 3.6(b) shows an elastic joint where both beam and column are not at high stresses. Figures 3.6(c) and (d) display joints where plastic hinges occur in the column

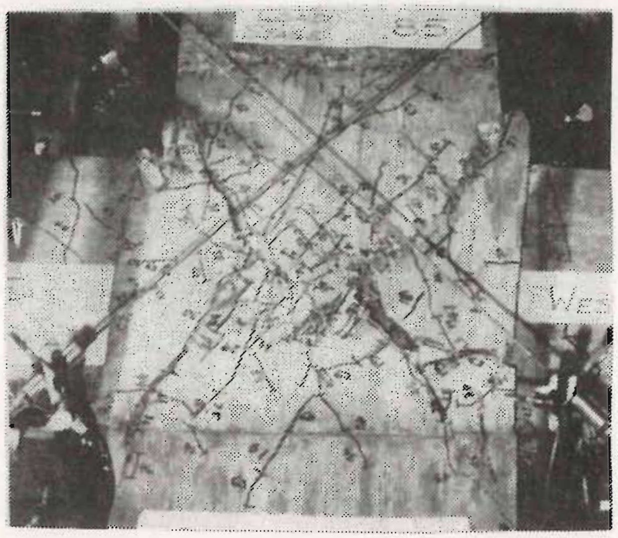


Fig. 3.1 Compression failure of a joint with hinges under cyclic loading (Meinheit and Jirsa, U.S.A.)

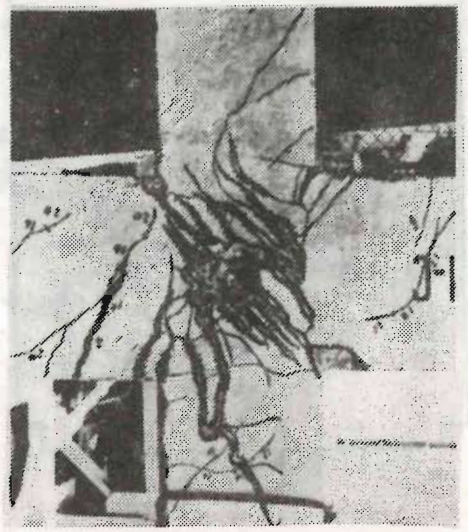


Fig. 3.2 Compression failure of a beam-column joint without hinges under skew-symmetric loading (Vasiliev, et al, U.S.S.R.)

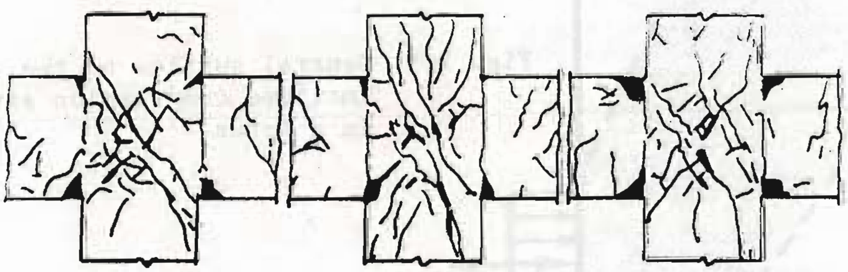


Fig. 3.3 Compression failure of beam-column joints after formation of hinges under cyclic loading (Kamimura, et al, Japan)

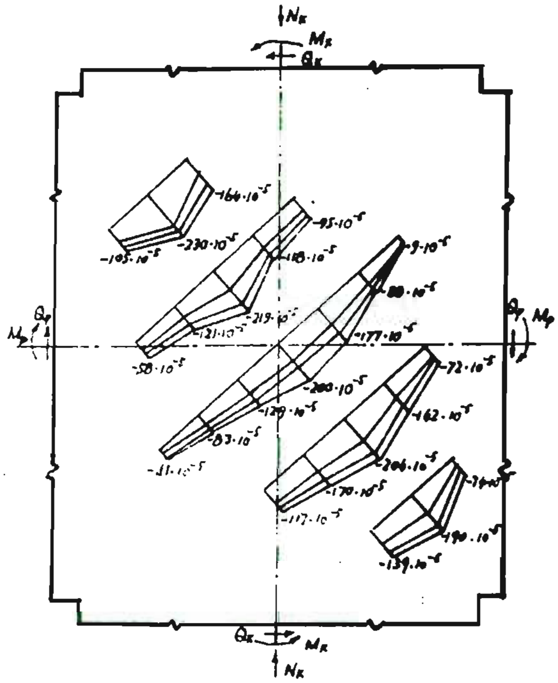


Fig. 3.4 Distribution of principal compressive deformations in the central zone of a beam-column joint (A. P. Vasiliev, et al, U.S.S.R.)

Note: The greatest values are given in the plots by the curves obtained at the load close to the ultimate load (95%)

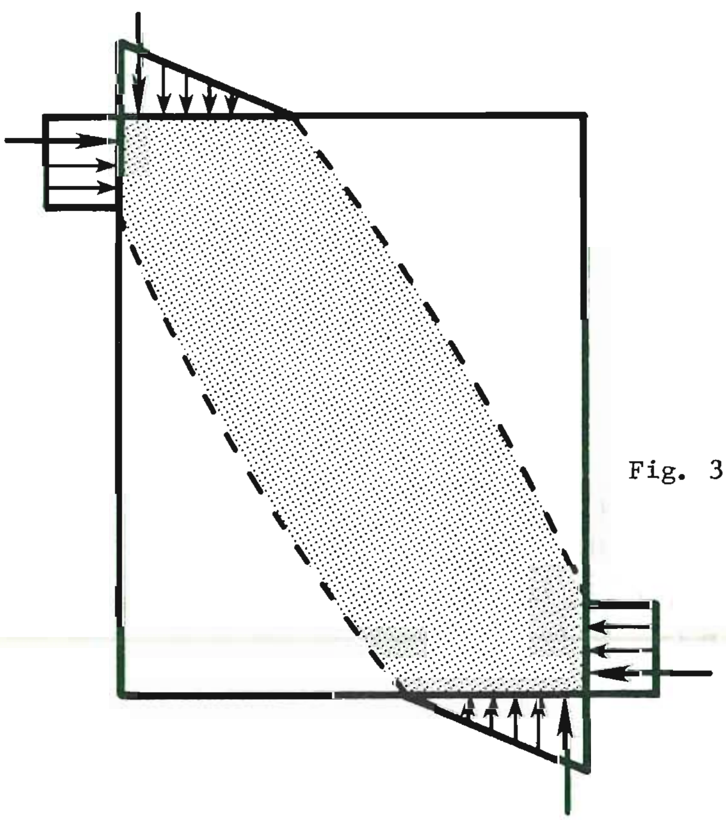
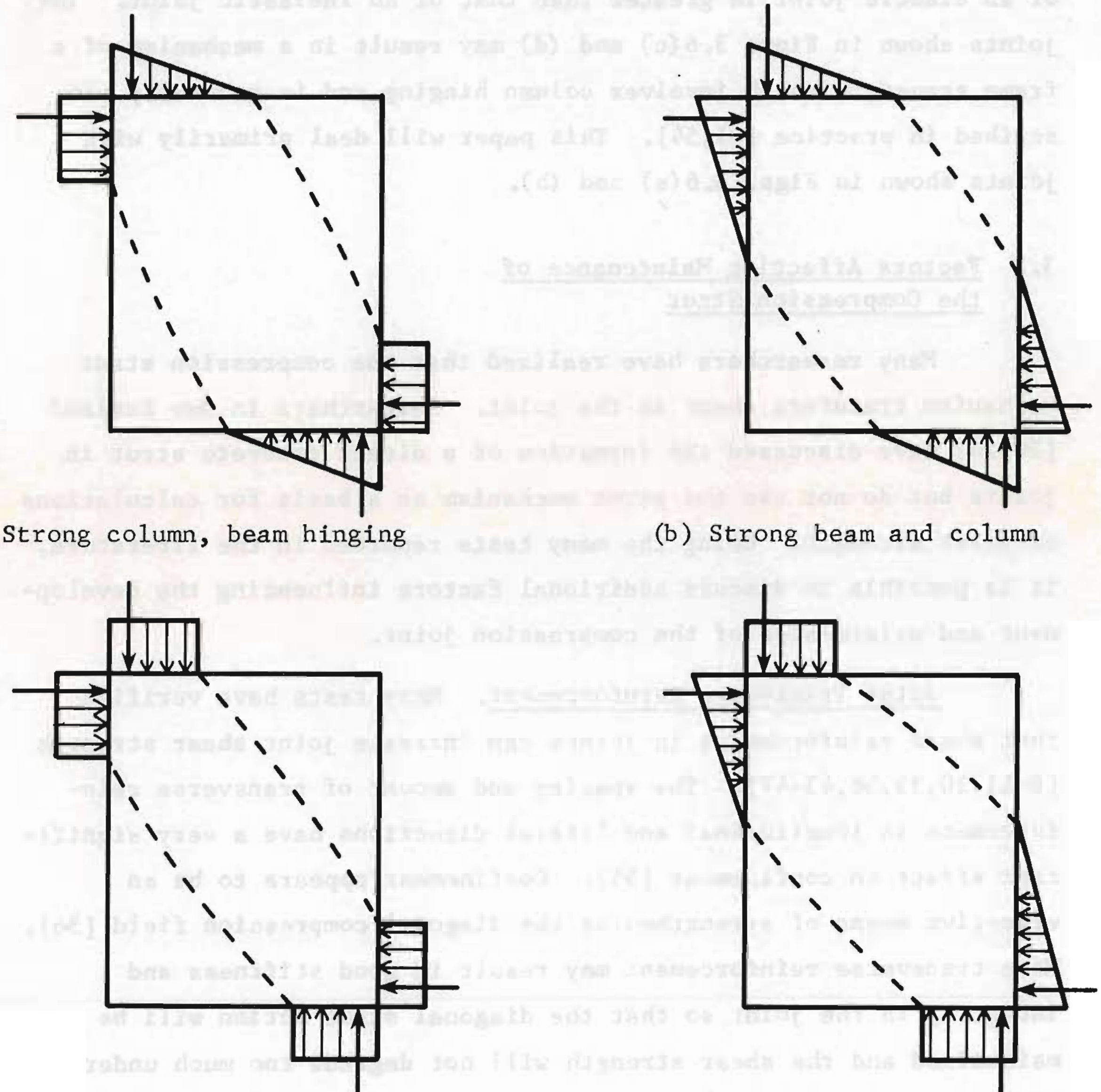


Fig. 3.5 General outline of the inclined compression strut in a joint



(a) Strong column, beam hinging

(b) Strong beam and column

(c) Beam and column hinging

(d) Weak column, strong beam

Fig. 3.6 Strut orientation

sections adjacent to the joint. Because the joint shown in Fig. 3.6(b) is bounded by larger compressive zones, the cross section of the equivalent compressive strut is larger than that of the joint shown in Fig. 3.6(a). This may explain partially why the shear strength of an elastic joint is greater than that of an inelastic joint. The joints shown in Figs. 3.6(c) and (d) may result in a mechanism of a frame structure which involves column hinging and is generally proscribed in practice [51,54]. This paper will deal primarily with joints shown in Figs. 3.6(a) and (b).

3.3 Factors Affecting Maintenance of the Compression Strut

Many researchers have realized that the compression strut mechanism transfers shear in the joint. Researchers in New Zealand [20-24] have discussed the formation of a direct concrete strut in joints but do not use the strut mechanism as a basis for calculations of joint strength. Using the many tests reported in the literature, it is possible to discuss additional factors influencing the development and maintenance of the compression joint.

Joint Transverse Reinforcement. Many tests have verified that shear reinforcement in joints can increase joint shear strength [8-11,30,33,36,43-47]. The spacing and amount of transverse reinforcement in longitudinal and lateral directions have a very significant effect on confinement [55]. Confinement appears to be an effective means of strengthening the diagonal compression field [56]. More transverse reinforcement may result in good stiffness and integrity in the joint so that the diagonal strut action will be maintained and the shear strength will not degrade too much under cyclic loading. In this case, the number of diagonal shear cracks increases and the crack widths are smaller (Fig. 3.7) than those with small transverse reinforcing ratios [11]. Reports from the U.S.S.R. [45,46] clearly illustrate the strut mechanism and the influence of joint reinforcement. Specimens reinforced with the welded mesh failed at the face of the joint rather than by crushing along the joint diagonal (Fig. 3.8).

Beam Hinging. It has been observed in tests that when the joint core fails, the compression zone in the beam hinges adjacent to the column faces spalls away. The crushing may be attributed to high strain in the tensile steel which produces a small compression zone in the beam. After checking the results of over 100 specimens in which beam hinging occurred (Chap. 4), it was found that the diagonal compression strut model was still valid in determining the shear resistance of the joint. It seems that joint shear may be resisted by diagonal strut action even though plastic hinges form in the beams close to the joint.

Bond in the Joint. With cyclic loading, bond loss progresses from the beam face into the joint. Strains well in excess of yield may occur within the joint. When bond is lost, the ability of the steel to transfer stress to the joint concrete is diminished. Under this condition, some researchers have questioned the continued existence of the strut. However, Hamada [25,26] and others in Japan have shown that at ultimate load the compression strut resultant is not related to the bond of the beam bars in the joint (Fig. 3.9). Although there was much difference in the bond strength between deformed bars and paraffin-coated bars used in the test, the strut forces C were nearly the same at ultimate load, as shown in Fig. 3.9. Fenwick [71,72] in New Zealand indicates that the diagonal strut mechanism does not depend on bond in the joint zone but it does require that the bars be anchored on the far side from the flexural tension force. There are a number of specimens reported in which bond failure occurs in the joints. Therefore, it appears that shear resistance of the concrete strut may be considered regardless of the degradation of bond along the boundaries of the joint, provided the bars can be anchored adequately outside the joint.

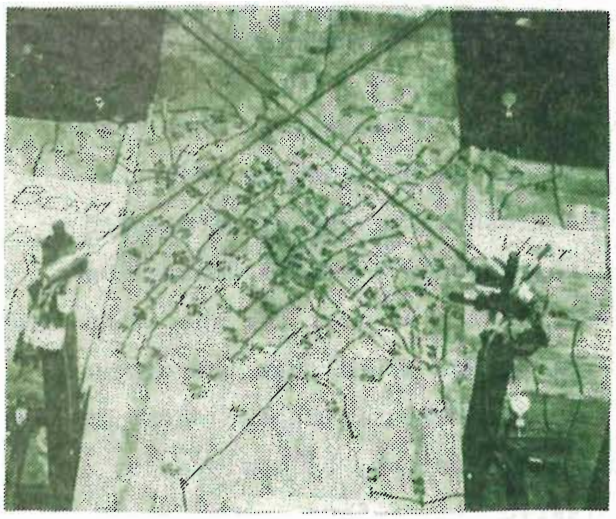


Fig. 3.7 Joint cracking pattern for specimen with high joint reinforcement ratio (Meinheit and Jirsa, U.S.A.)

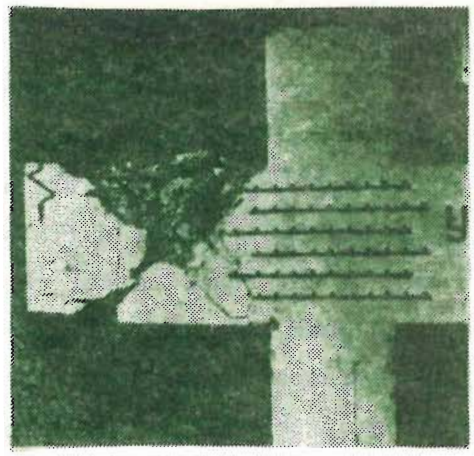


Fig. 3.8 Failure of a test specimen at the connection of a beam to the column when the central zone of the joint is reinforced with mesh (A. P. Vasiliev, et al, U.S.S.R.)

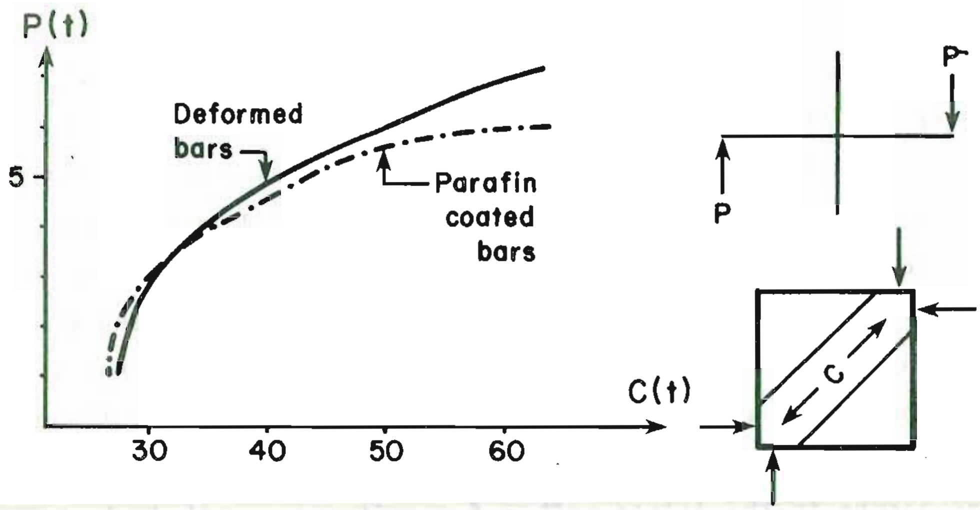


Fig. 3.9 Load versus strut resultant (Hamada et al, Japan)

3.4 Mechanism for Transfer of Joint Shear Force

The purpose of this study is to develop a model to be used in calculations for joint strength. As mentioned before, the modified beam shear mechanism and the truss mechanism may not always be good models for calculations of strength. It is likely that a reasonable model will involve a combination of contributions from both the concrete and transverse reinforcement. A model based on only one of the materials is likely to oversimplify the problem. In this report, the model for joint shear will be based on the use of a concrete compression strut. Other factors such as transverse reinforcement, lateral beams, and type of loading, will be evaluated in terms of their influence on the strength of the compression strut.

CHAPTER 4

MONOTONIC SHEAR STRENGTH OF JOINT--APPLICATION OF INCLINED STRUT

The resistance of a beam-column joint to shear forces has been postulated to be developed by a concrete compression strut. The strength of the mechanism depends on a variety of parameters [11, 57,58] which will be discussed individually. Based on the equilibrium of a fairly simple mechanical model, the equations for strength under monotonic loading will be developed in this chapter and used as a function for that under cyclic loading.

4.1 Definition of Monotonic Shear Strength

Monotonic shear strength is defined as the maximum strength that the joint can reach under steadily increasing load to joint failure. It can be determined at the peak of the first half of the first cycle soon after yielding, as shown in Fig. 4.1. It may also be determined from the envelope of peak shears reached in each cycle of a repeated load history. The shear strength is reached when the concrete compression zone is crushed and longitudinal beam bars do not reach yield strength at the face of the column. Generally, the strength decreases with the increase of deformation or reversal of loading. In joints where beam flexural yielding occurs, the shear acting on the joint is obviously less than the shear capacity of the joint. Therefore, the measured capacity of the joint in specimens failing in flexure represents a lower bound on the shear capacity of the joint.

4.2 Basic Shear Strength Using Inclined Strut

Equivalent Strut. As discussed in Chapter 3, the shape of the strut is something like a shuttle. To simplify the calculations

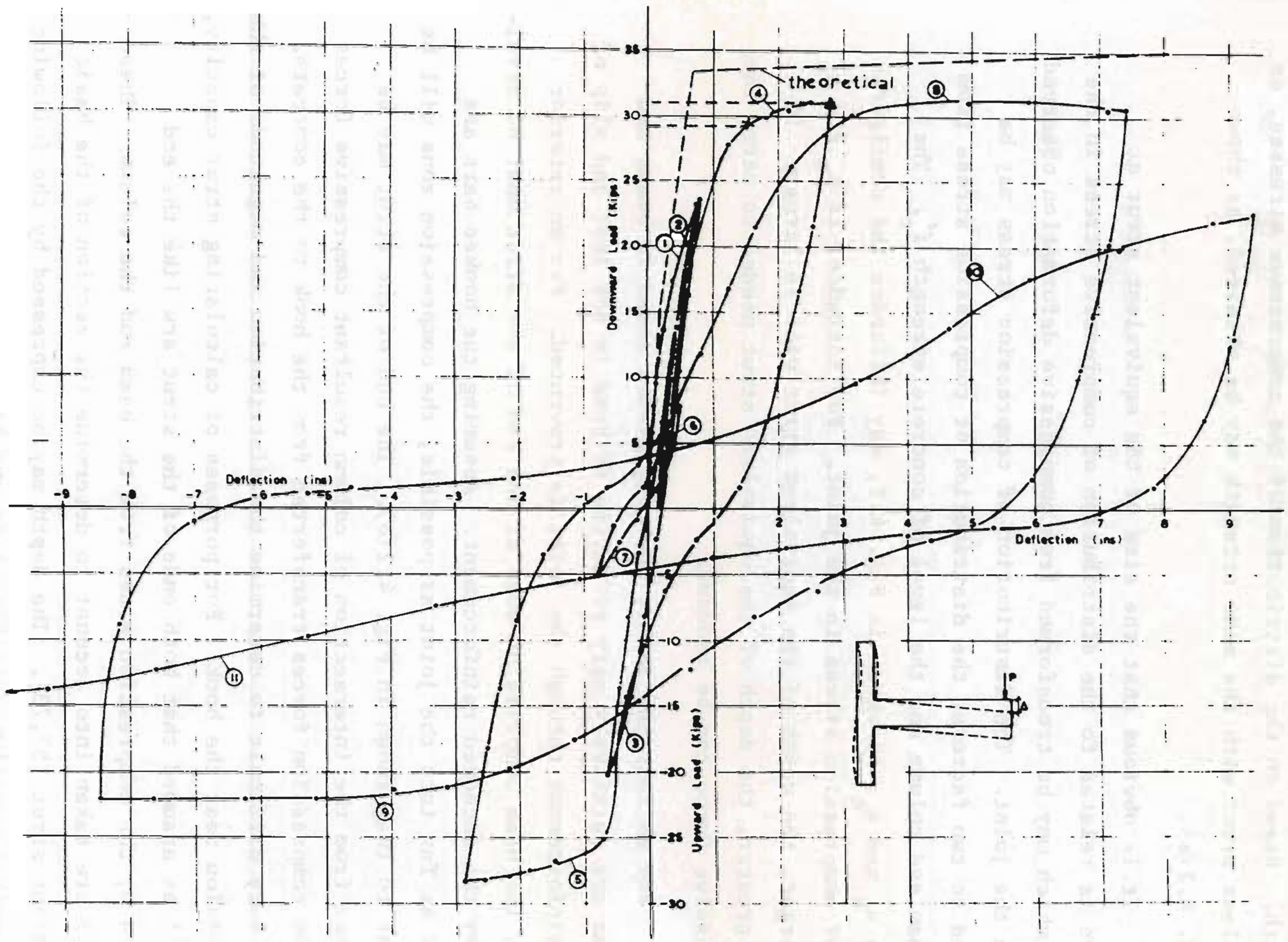
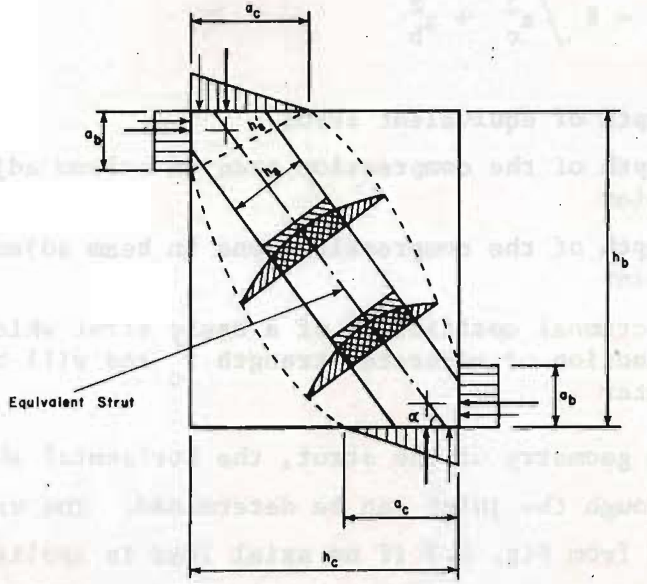


Fig. 4.1 Load-deflection curve under cyclic loading (Renton, New Zealand)

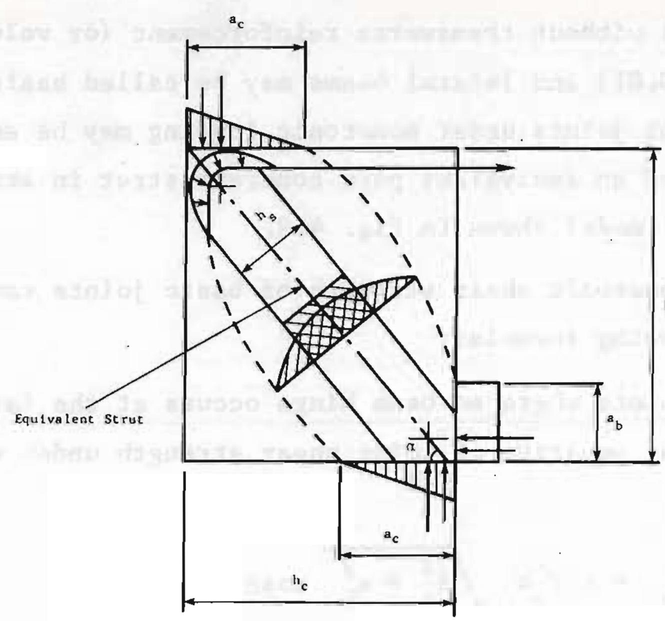
for such a strut, the concept of an equivalent strut is adopted so that calculations are similar to those for axially loaded column strength. Based on the distribution of the compressive stresses, an equivalent strut with the same strength may be obtained, as shown in Fig. 4.2(a).

It is obvious that the size of the equivalent strut at failure is related to the distribution of compressive stress in the joint which may be transformed from compressive deformation observed within the joint. The distribution of compression stress may be related to two factors, the distribution of compression stress from the beam and column and the level of concrete strength f'_c. The values a_b and a_c, as shown in Fig. 4.2, may influence the configuration of compression stress in the joint. For instance, if a_b and a_c are larger, the depth of the equivalent strut will be larger. If f'_c is greater, the depth of the equivalent strut needed to carry the compressive force can be reduced.

For an interior joint, the compression zones in beams and columns are relatively easy to define if bond is not lost and slip of the reinforcement through the joint is prevented. For an exterior joint, the beam compression zone at one end of the strut must be developed by the anchored reinforcement. Assuming the hooked bars are placed as far into the joint as possible, the compression zone will be similar to that shown in Fig. 4.2(b). The end of the strut may be obtained from the intersection of column resultant compressive forces and the compression forces transferred from the hook to the concrete. It is very difficult to determine the distribution and magnitude of the compression near the hook. For purposes of calculating strut capacity, it will be assumed that both ends of the strut are like the end defined by the compression zones from the beam and the column. These factors are taken into account to determine the section of the basic equivalent strut [57,73]. The depth may be expressed by the following experimental equation (see Fig. 4.2a and b):



(a) Interior joint



(b) Exterior joint

Fig. 4.2 Principal compressive stresses, location and size of an equivalent strut

$$h'_s = \sqrt{a_c^2 + a_b^2}$$

$$h_s = K\sqrt{a_c^2 + a_b^2} \qquad (4.1)$$

where h_s = depth of equivalent strut

a_c = depth of the compression zone in column adjacent to the joint

a_b = depth of the compression zone in beam adjacent to the joint

K = sectional coefficient of a basic strut which is a function of concrete strength f'_c and will be discussed later

By defining the geometry of the strut, the horizontal shear force transferred through the joint can be determined. The value of a_c can be obtained from Fig. 4.3 if no axial load is applied to the column, or from Fig. 4.4 if bending and axial load are applied. Figure 4.3 may be used for a_b if the beam does not develop hinging at the face of the joint.

Joints without transverse reinforcement (or volumetric percentage $\rho_s \le 0.01$) and lateral beams may be called basic joints. The strength of joints under monotonic loading may be estimated by the strength of an equivalent pure concrete strut in accordance with the analytical model shown in Fig. 4.2.

The monotonic shear strength of basic joints can be determined from the following formulas:

For joints where no beam hinge occurs at the face of the column (Fig. 4.5), the equation of joint shear strength under monotonic loading is

$$Q_m = K f'_c b_c \sqrt{a_c^2 + a_b^2} \cos\alpha \qquad (4.2)$$

where b_c = width of the joint, that is, width of the column, in.

α = inclined angle of the equivalent strut

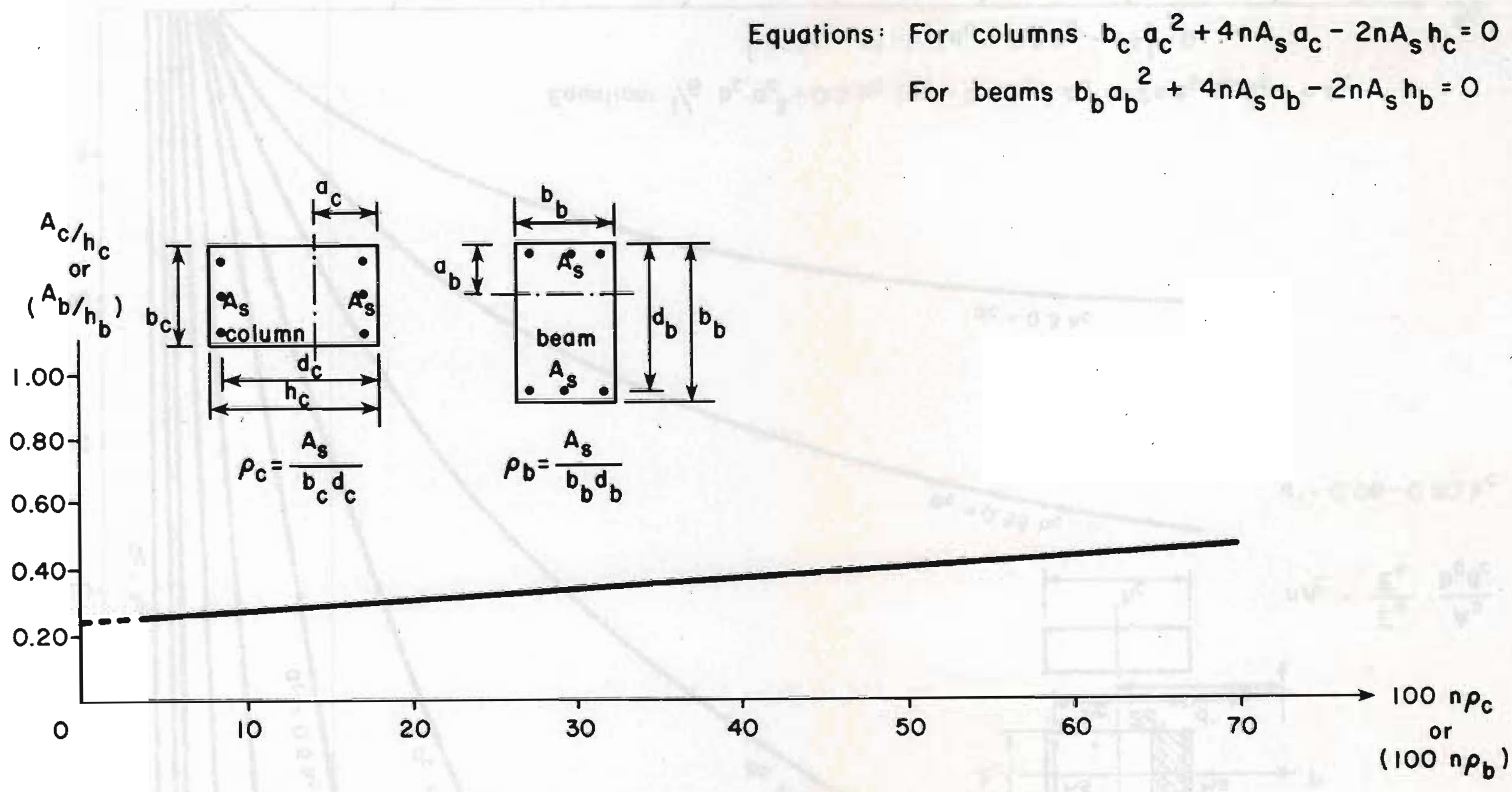


Fig. 4.3 Depth of compression zone a_c or a_b at faces of joints, no axial load on member, no hinging

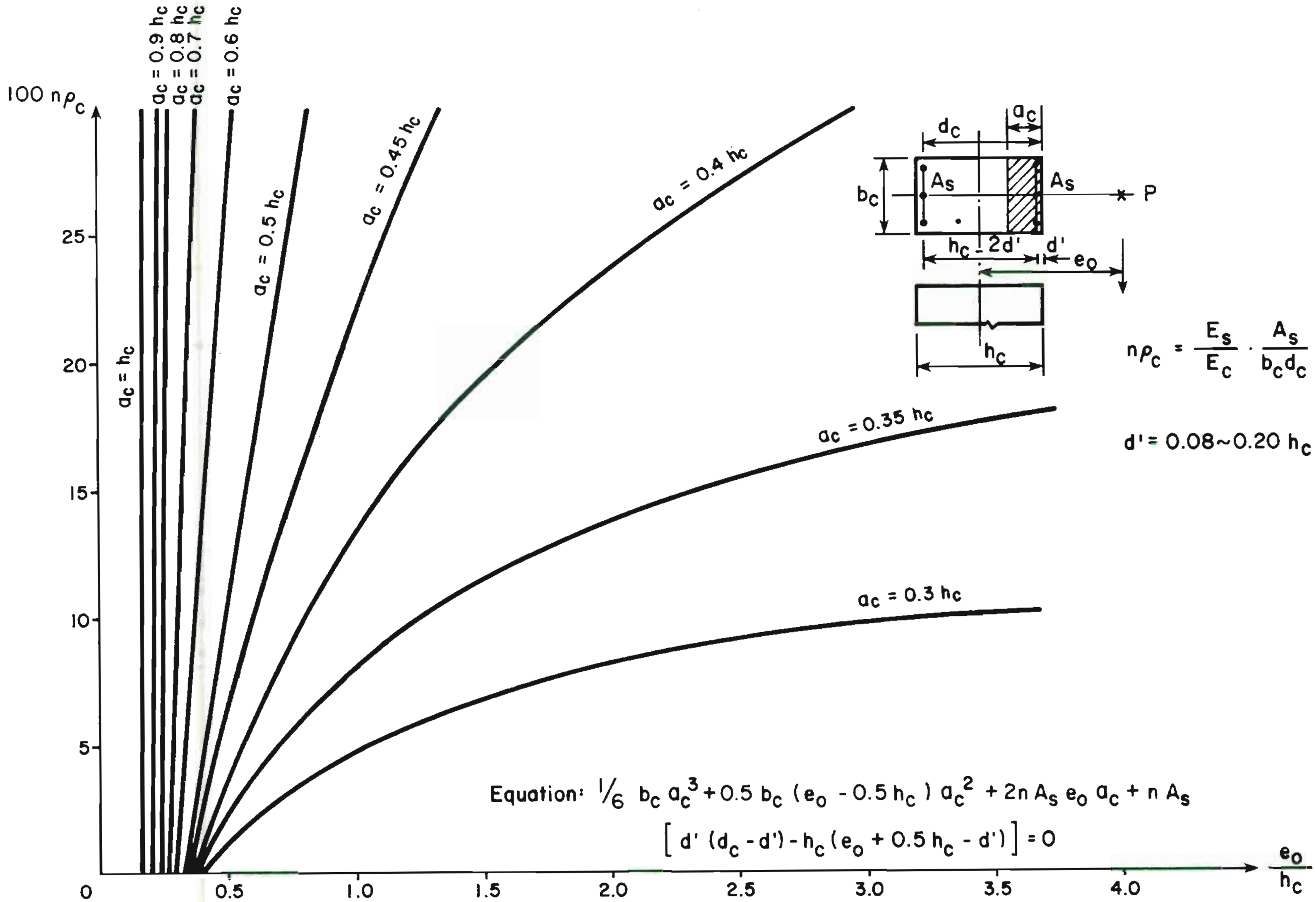


Fig. 4.4 Depth of compression zone a_c at face of the joint, with axial load, no hinging

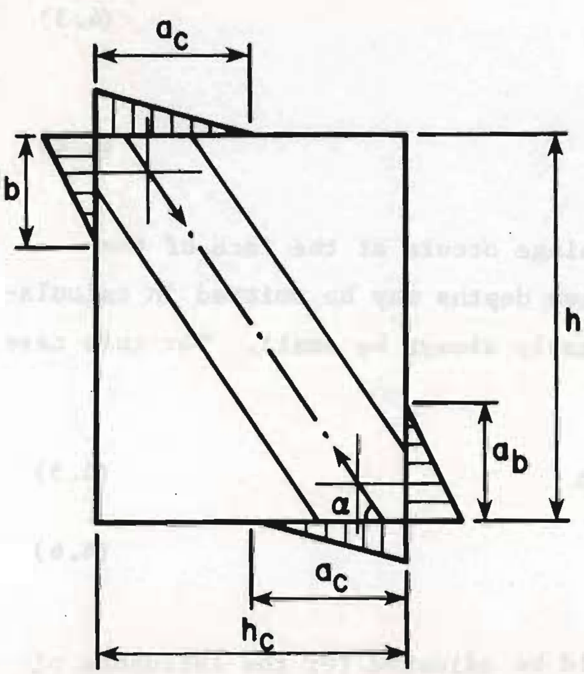


Fig. 4.5. Terms needed for calculation of strength of non-hinging joints at compression strut failure

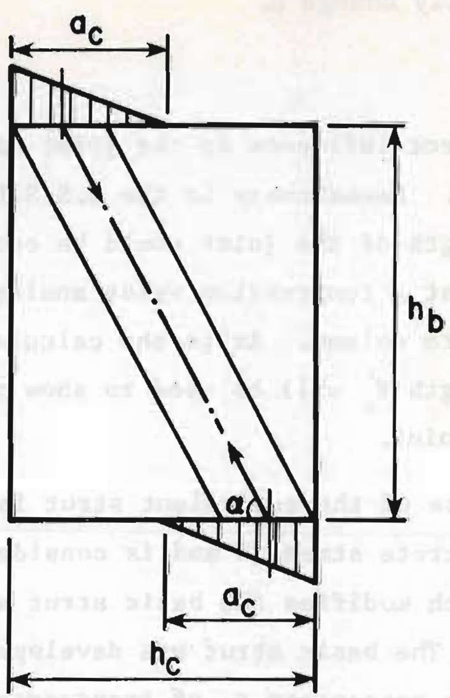


Fig. 4.6. Terms needed for calculation of strength of hinging joints at compression strut failure.

$$\cos \alpha = \frac{1}{\sqrt{1 + \tan^2 \alpha}} \quad (4.3)$$

$$\tan \alpha = \frac{h_b - 2/3\ a_b}{h_c - 2/3\ a_c} \quad (4.4)$$

For joints where a beam hinge occurs at the face of the column (Fig. 4.6), beam compressive depths may be omitted in calculations because such depths will nearly always be small. For this case, the equation is as follows:

$$Q_m = K\ f'_c\ b_c\ a_c \cos \alpha \quad (4.5)$$

and

$$\tan \alpha = \frac{h_b}{h_c - 2/3\ a_c} \quad (4.6)$$

In Fig. 4.6, the value of α should be adjusted for the influence of the bend location of the hooked bar, as shown in Fig. 4.2(b). However, this adjustment will not substantially change α.

4.3 Concrete Strength

Concrete strength has a direct influence on the joint shear strength using the strut mechanisms. Researchers in the U.S.S.R. [45,46] demonstrated that the strength of the joint could be controlled by failure of the concrete at a compression value analogous to that of an axially loaded concrete column. As in the calculation for column strength, concrete strength f'_c will be used to show that concrete fails by crushing in the joint.

As mentioned before, the size of the equivalent strut is assumed to be a function of the concrete strength and is considered by using a sectional coefficient K which modifies the basic strut section and produces the equivalent strut. The basic strut was developed using test results in which the volumetric percentage ρ_s of transverse reinforcement is low, $0 \leq \rho_s \leq 0.01$. With a small amount of transverse reinforcement, concrete in the joint has to resist most (if not all) of the load applied to the boundaries. The data in Fig. 4.7 were used to obtain an empirical equation for the equivalent strut factor K. The equation is as follows:

$$K = 1.20 - 0.1f'_c \quad (f'_c \text{ in ksi}) \tag{4.7}$$

Data in Fig. 4.7 show that sectional coefficient K is nearly linear with concrete strength f'_c.

The expression for K $(1.20 - 0.1f'_c)$ shows that joint shear strength, which is a function of K f'_c does not increase linearly with concrete strength. Recent tests conducted at The University of Texas at Austin [74] confirm this approach. Two specimens were identical except for concrete strength, 2800 and 4500 psi. Although the concrete strength was 61% greater for one specimen, the maximum shear increased by less than 15%. Therefore, increasing concrete strength is unlikely to increase joint shear strength markedly.

In addition, high concrete strength can enhance bond strength so that yield penetration of reinforcement may be reduced. However, poor bond due to low concrete strength may not reduce strut capacity as discussed earlier.

4.4 Column Axial Load

Some investigators [8-10,20-24,56,57] consider that joint shear strength is affected by the magnitude of the column load and that greater column load can result in higher shear strength. Some codes [51,52,54,59,60] include the effect of axial load on joint shear strength in design equations. As the compressed area of concrete in the column section above or below a joint increases due to increasing axial load, the depth of compression strut will increase. Moderate column loads may increase the size of the strut so that it can transfer more shear force through the joint.

It should be noted that small column axial load in conjunction with large forces on the joint from the beam may produce a small strut. Such a combination constitutes a critical load case that should be considered in design. Where the column load is high, approaching axial compressive failure, joint shear resistance will be small

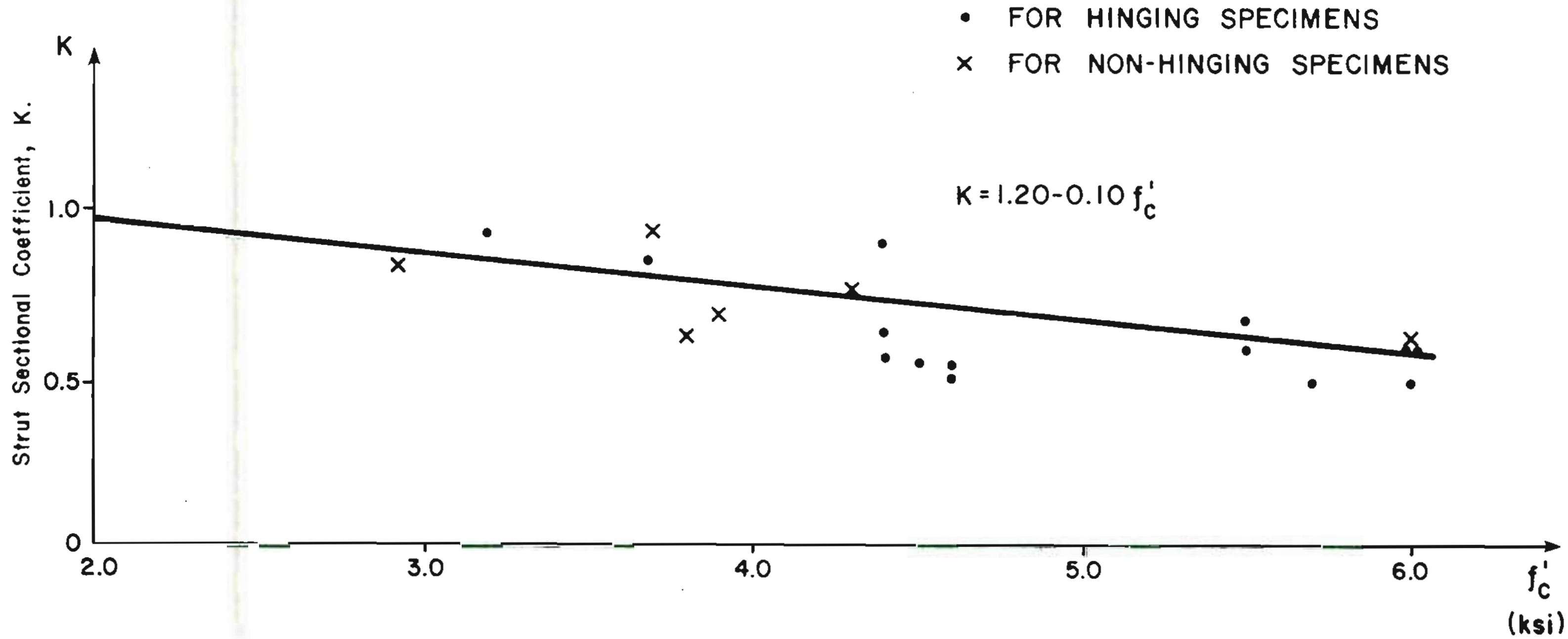


Fig. 4.7 Sectional coefficient K of strut versus concrete strength f_c', $\rho_s \leq 0.01$

because the strut capacity will be utilized almost entirely in carrying the axial load.

The level of column axial load has a strong influence on the strut inclined angle α. The higher the column load, the greater the inclined angle. Some test data from New Zealand indicate that the inclination α of diagonal cracks may not be 45° when joint failure occurs [16-24]. Column load may also increase the bond strength of beam bars through the joint and may slow decay in strength and stiffness with cycling [62].

4.5 Geometric Parameters

In addition to the depth of the equivalent strut, as mentioned above, the column width b_c determines the cross-sectional area of the strut. The ratio of the beam width b_b to the column width b_c may have some influence on the effectiveness of the joint strut mechanism. When the ratio is smaller, it is unlikely that the strut will involve the entire width b_c of the joint, as shown in Fig. 4.8. From an analysis of a number of tests of specimens with ratios of b_b to b_c less than 0.75, the value of $(b_b + b_c)/2$ appears to give a reasonable equivalent strut. In this case, the area over which the compressive force is transferred in the lateral direction, that is, the width of the compression strut, is neither the beam width b_b nor the column width b_c, but may lie between the two values. Therefore, the mean value may be used to evaluate the strut strength until further test data become available.

Joints with beams wider than columns have been investigated very little so far. Recently, the comparative tests on the influence of beam size on joint confinement have been carried out at The University of Texas at Austin. The test results [74] indicated that existence of cracks at the face of the column may reduce or destroy joint confinement by beams. In this case, joint confinement should not be considered in design even though the size of framing beams is large. The column above and below the joint region exhibited considerable compressive shear spalling. The spalling reduced the column section greatly. The results confirm the need for transverse

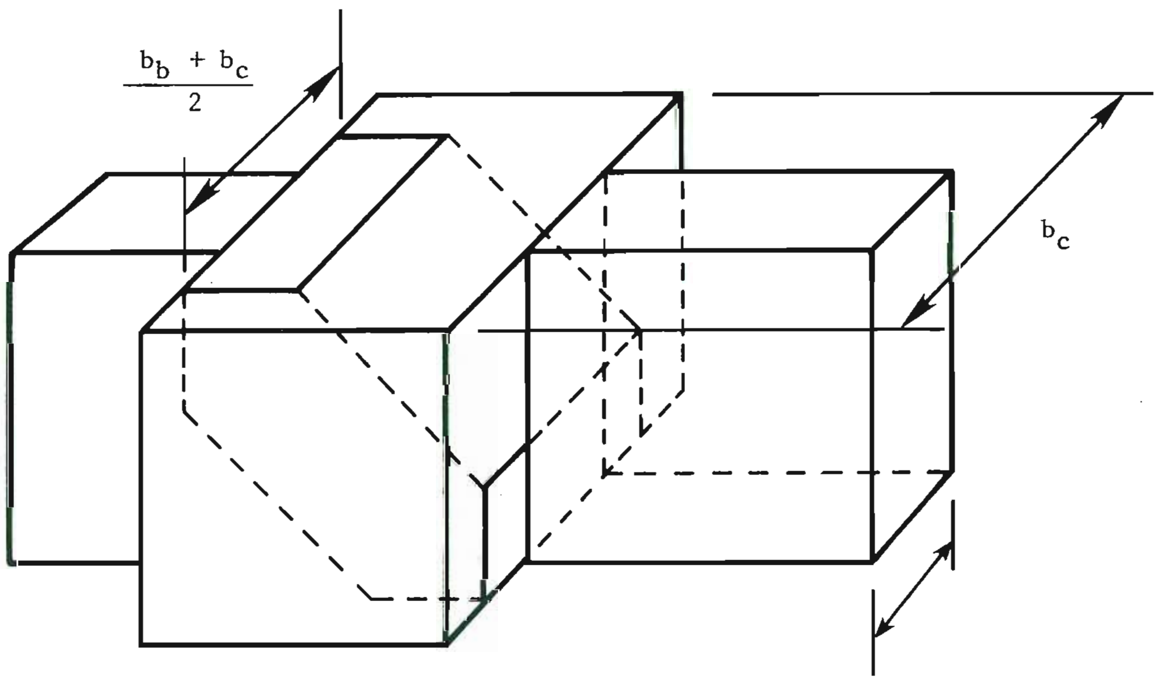


Fig. 4.8 Concrete strut in joint with beams narrower than column

reinforcement to continue into the column above and below the joint. The inclination α of the strut to the horizontal may be obtained approximately by the line between the intersections of concrete compressive forces in the beam and column at diagonally opposite corners, as shown in Fig. 4.2. In general, the angle of the strut at ultimate load is the same as the inclination of cracks at cracking load [26].

4.6 Transverse Reinforcement

There is general agreement that transverse reinforcement improves the shear capacity of joints. However, the manner in which the reinforcement contributes to the shear capacity is still not well understood. Data from a large number of tests [8-11,18,19,23,30,36, 43-46] indicate that the joint reinforcement provides confinement and shear resistance to the joint and increases the anchorage characteristics of beam bars in the joint.

Some researchers [11,18] have pointed out that the greater the amount of shear reinforcement in the joint, the greater the shear strength, but the increase in shear strength is not proportional to the amount of shear reinforcement. Experimental investigations have demonstrated that reasonable shapes and arrangements of transverse reinforcement could enhance the compression-carrying capacity of the concrete in the core of an inelastic joint under large deformations [23,30,45,46]. Lateral reinforcement or welded mesh in closely knit cages can be used to increase the efficiency of confinement and ductility of the concrete joint core [55], thus improving the performance of the joint in shear.

Based on a review of test data, it appears that transverse reinforcement acts primarily to restrain lateral expansion of the compressive strut. As a result the influence is rather small and indirect--only an excessively large volume of transverse reinforcement will substantially increase the capacity of the strut. An analysis of joints without lateral beams indicates that the increase in joint shear strength is approximately linear with the increase in transverse reinforcement beyond some minimum volumetric ratio (Fig. 4.9). The empirical equation for ζ which reflects the effect of transverse reinforcement on shear strength is as follows:

$$\zeta = 0.95 + 4.5 \rho_s \qquad (4.8)$$

where ρ_s = volumetric percentage of transverse reinforcement

$$= \frac{(mb^* + nh^*) A_t}{s_t b^* h^*} \qquad (4.9)$$

A_t = area of the joint hoop bar or the wire of welded mesh (one bar or one wire area), in.^{2}

b^* = joint core dimension to outside of hoop or end of wire in y direction, in. (Fig. 4.10)

h^* = joint core dimension to outside of hoop or the end of wire in x direction, in. (Fig. 4.10)

s_t = spacing of joint transverse reinforcement, in.

m = number of hoops and single leg ties or wires in y direction

n = number of hoops and single leg ties or wires in x direction

For rectangular column with m = n

$$\rho_s = \frac{(b^* + h^*)}{h^*} \frac{n A_t}{b^* s} = \frac{(b^* + h^*)}{h^*} \rho_w \qquad (4.10)$$

For square column m = n

$$\rho_s = 2 \frac{n A_t}{b^* s_t} = 2\rho_w \qquad (4.11)$$

Note that Eq. (4.8) is suitable for $0.01 \leq \rho_s \leq 0.06$ according to the available data. Tests in U.S.S.R. indicate that the effect of welded wire mesh is much better than hoops. However, it may be difficult to fabricate in-situ. In order to obtain an upper bound for ζ, those pairs of specimens which had the same conditions except different amounts of transverse steel were selected, one

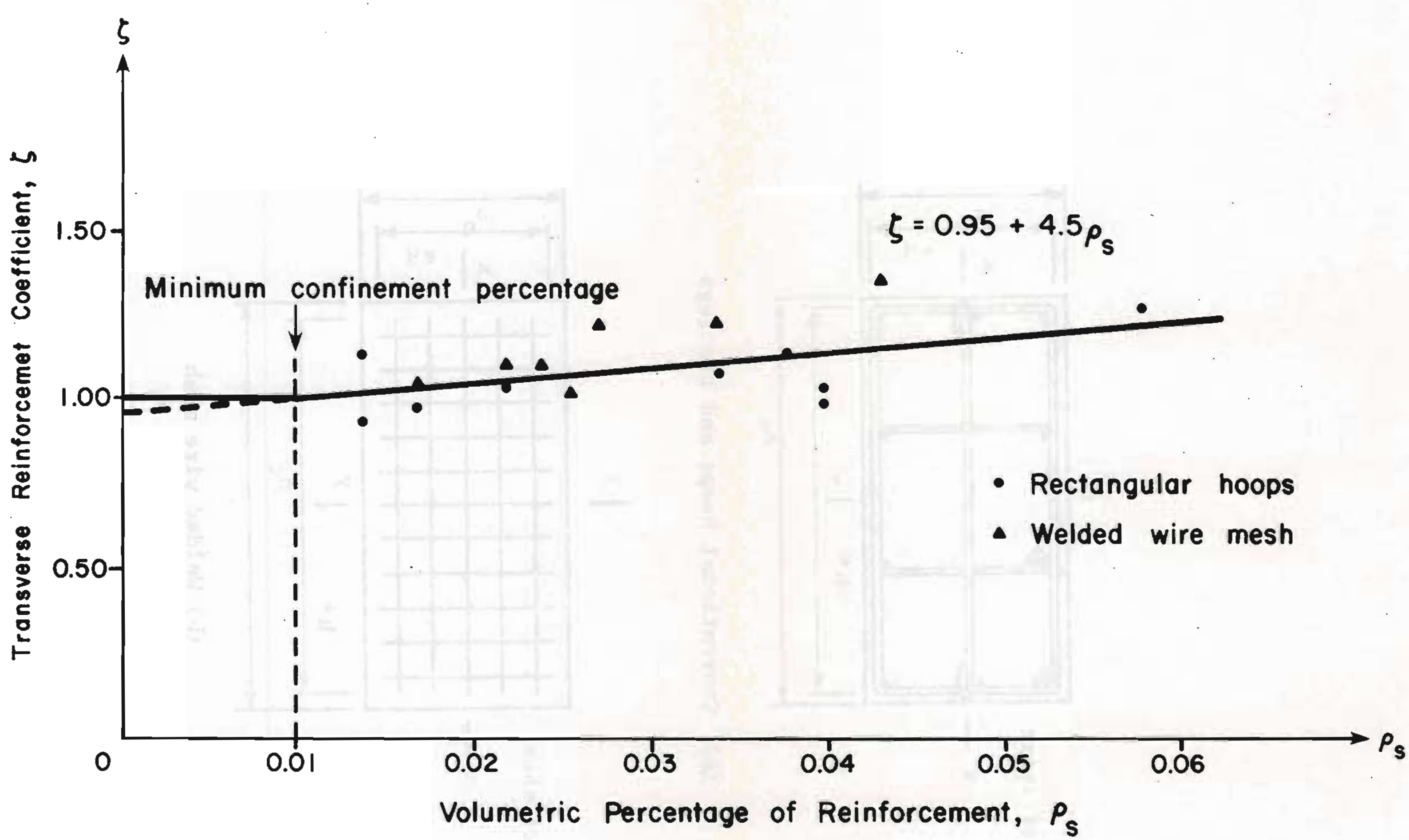
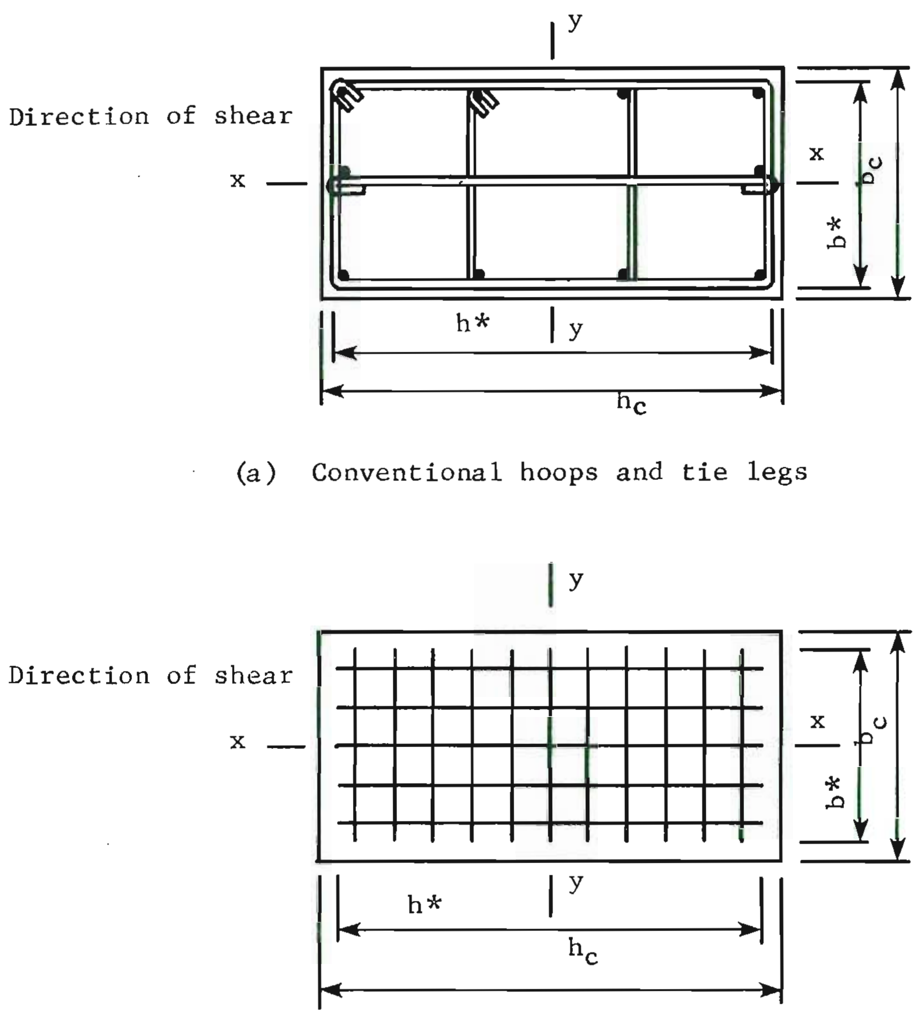


Fig. 4.9 Coefficient ζ for effect of transverse reinforcement (without lateral beams)



(a) Conventional hoops and tie legs

(b) Welded wire mesh

Fig. 4.10 Geometry of transverse reinforcement with $\rho_s > 0.01$, the other with $\rho_s \le 0.01$. The ratio of test values Q_m^t of joint shear strength with $\rho_s > 0.01$ to test values Q_o^t with $\rho_s \le 0.01$ versus factor ζ reflecting the effect of transverse reinforcement, is shown in Fig. 4.11. From the figure it can be seen that maximum shear strength with transverse reinforcement is about 1.25 times as much as shear strength without transverse reinforcement ($\rho_s \le 0.01$) where $\rho_s = 0.055$. Therefore, 1.20 is taken as upper bound of ζ in design. It appears that more steel is unlikely to increase shear capacity. Meanwhile, other problems such as congestion, clearance, or fabrication would keep the volumetric steel ratio smaller. Typical manageable values of ρ_s are in the 0.03 to 0.04 range.

4.7 Lateral Beams

Lateral beams are considered to confine the joint only when beams are present on both sides of the joint and are not loaded either flexurally or axially. Many tests have verified that lateral beams, perpendicular to the direction of applied shear, may aid confinement of the joint core so that joint shear strength increases [8-11, 27-29,32,34]. Japanese researchers conducted some comparative tests on the effect of lateral beams [27-29] and found that crushing along the joint diagonal occurred in the joints with no lateral beams or a lateral beam on one side only. The shear capacity of joints with lateral beams on two sides increased markedly when compared with other cases. However, only when the ratio of the lateral beam width W_L to the total column depth h_c reaches a certain value can the effect of lateral beams be taken into account.

In a joint confined by lateral beams the total shear provided by the concrete compressive strut increases linearly with the ratio of the beam width W_L to the column depth h_c (Fig. 4.12). The influence of lateral beams on shear strength can be expressed as follows:

$$\gamma = 0.85 + 0.30\, W_L/h_c \qquad (4.12)$$

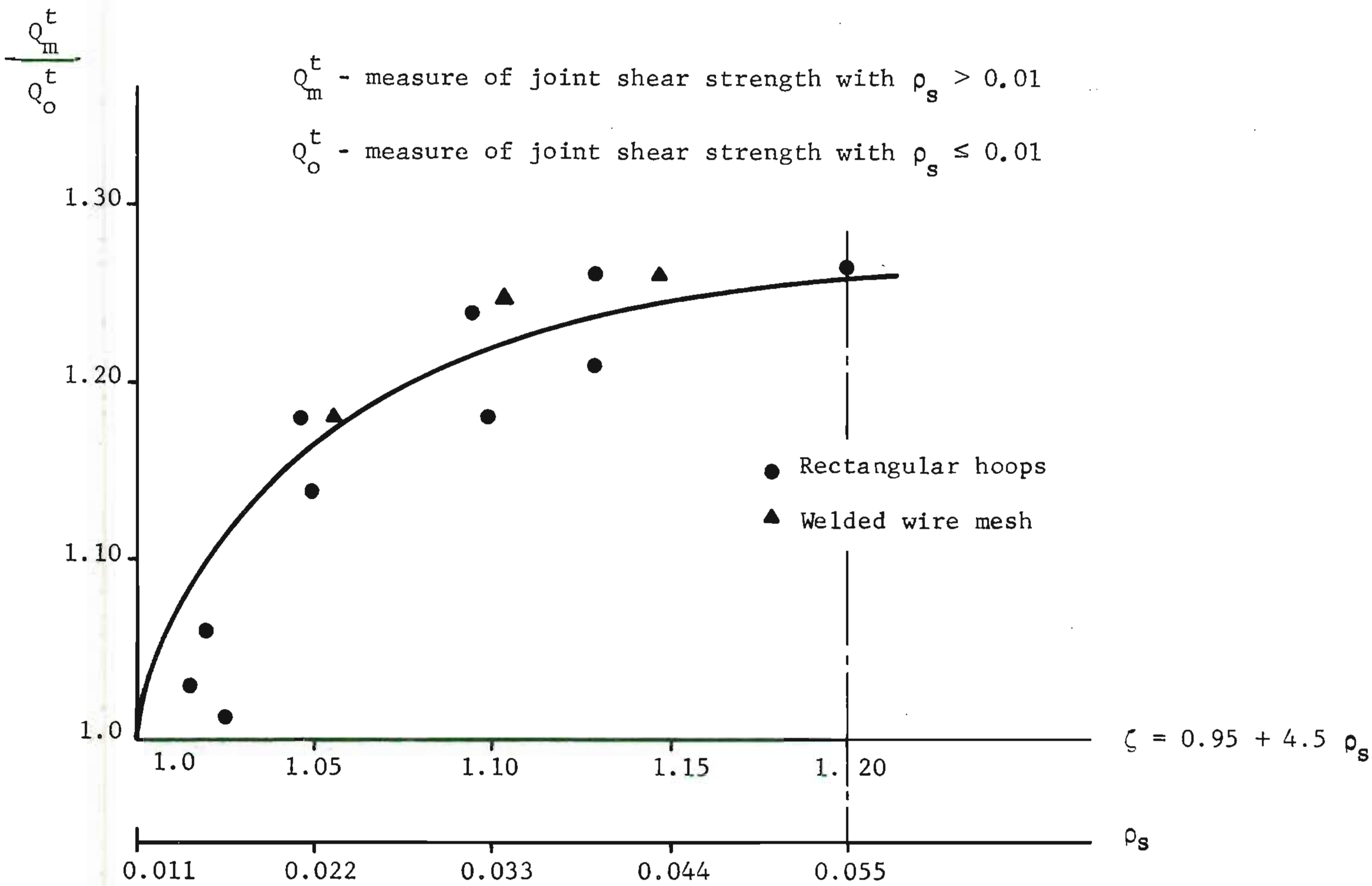


Fig. 4.11 Determination of maximum value of ζ for transverse reinforcement

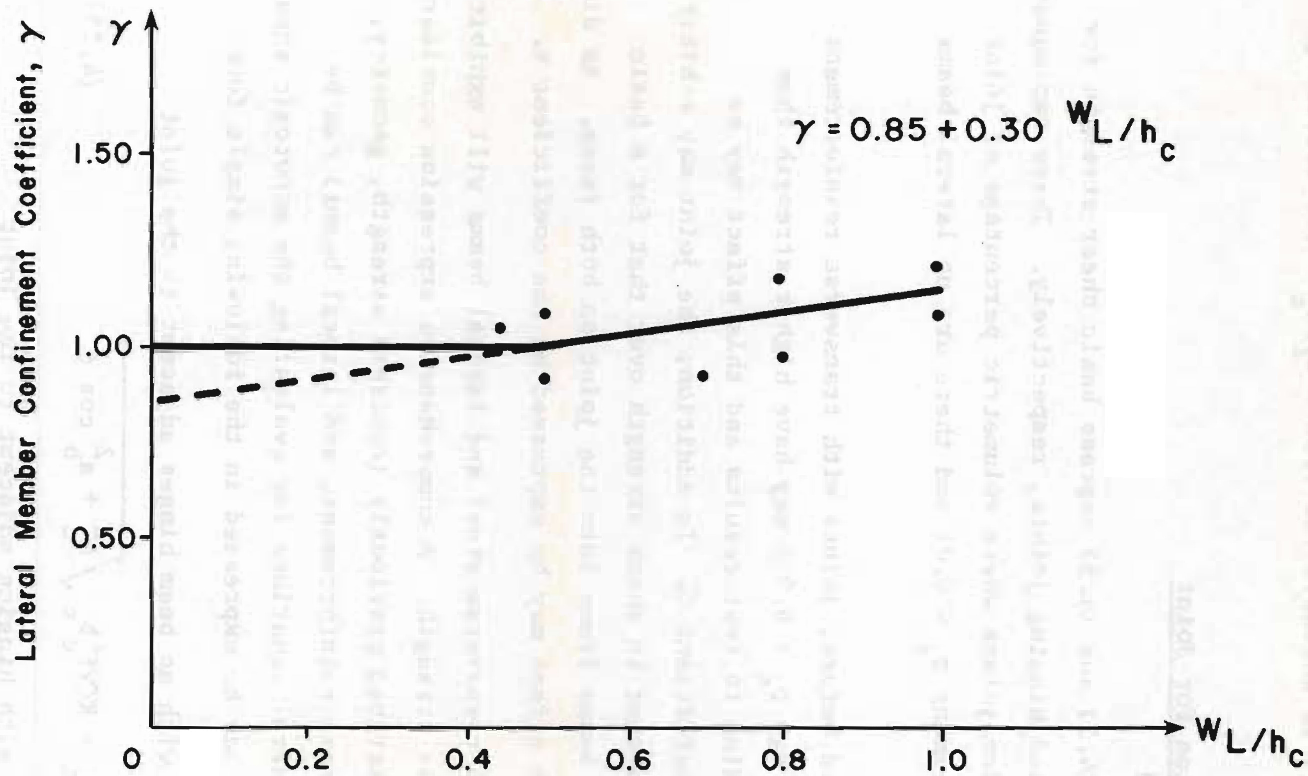


Fig. 4.12 Coefficient γ for effect of lateral beams in terms of the ratio of beam width W_L to column depth h_c

Note that Eq. (4.12) is suitable for $0.50 \leq W_L/h_c \leq 1.0$ according to the available data.

4.8 General Equation for Joint Shear Strength

Equations (4.2) and (4.5) express basic shear strength for nonhinging joints and hinging joints, respectively. These two equations can be used for joints where volumetric percentage of joint transverse reinforcement $\rho_s \leq 0.01$ and there are no lateral beams on two sides.

As mentioned before, joints with transverse reinforcement (volumetric percentage $\rho_s > 0.01$) may have higher strength than basic joints according to test results and this effect may be expressed by the coefficient ζ. In addition, the joint may exhibit considerable improvement in shear strength over that for a basic joint when lateral beams frame into the joint on both faces. As discussed before, this effect may be expressed by the coefficient γ.

Joints with transverse steel and lateral beams will exhibit an increase in shear strength. A comprehensive expression considering all factors described previously (concrete strength, geometry, axial load, transverse reinforcement, and lateral beams) can be developed. The general equations for evaluating the monotonic shear strength of joints may be expressed in the following simple form:

For joints with no beam hinges adjacent to the joint

$$Q_m = K\zeta\gamma f'_c b_c \sqrt{a_c^2 + a_b^2} \cos \alpha \qquad (4.13)$$

For joints with hinging adjacent to the joint

$$Q_m = K\zeta\gamma f'_c b_c a_c \cos \alpha \qquad (4.14)$$

where K = sectional coefficient of a basic strut obtained from Eq. (4.7)

ζ = coefficient reflecting the effect of transverse reinforcement obtained from Eq. (4.8)

γ = coefficient reflecting the effect of lateral beams obtained from Eq. (4.12)

The value of a_c may be obtained from Figs. 4.3 or 4.4, and the value of a_b from Fig. 4.3.

4.9 Comparison with Test Results

A large number of interior and exterior joints have been analyzed and the results summarized in Table 4.1.

TABLE 4.1 CALCULATED VS MEASURED RESULTS--MONOTONIC LOADING (Max. Strength)

Total Number of Specimens	Joints			Q_{test}/Q_{calc}	Standard Deviation
	Interior	Exterior	Subtotal	Average	
164	71	52	123 (F)	1.00	0.23
	36	5	41 (S)	1.10	0.14

Note: (F) - Flexural failure, hinges in beams formed prior to joint failure
(S) - Shear failure, beams did not reach yield

The measured peak strength for 164 specimens tested in the U.S., Japan, New Zealand, Canada, China, and the U.S.S.R. is tabulated in Appendix A. Predicted shear strengths using related equations and a comparison with test data are given. A large amount of information necessary to describe the specimen geometry, reinforcement, and type of failure has been included. The average and the standard deviation of the ratios of test values to calculated values for 41 specimens failing in shear are 1.10 and 0.14, respectively.

No attempt was made to determine whether specimens reported as failing in flexure might have failed in shear. Likewise, no attempt was made to calculate flexural capacity for specimens reported to have failed in flexure. In general, the information available in the literature precluded an evaluation of the bond loss through the joint. Such a phenomenon would have the effect of reducing the flexural capacity of the beams framing into the joint. It is interesting to note that for the 123 specimens reported failing in flexure, the average test to calculated shear capacity is 1.0, which indicates that the proposed approach represents a reasonable estimate of shear capacity. With a standard deviation for this group of 0.23, Eqs. (4.13) and (4.14) provide a conservative estimate of capacity when compared with flexural capacity, i.e., a designer using the compression strut approach is not likely to overestimate the shear capacity of the joint.

CHAPTER 5

SHEAR STRENGTH OF JOINTS UNDER CYCLIC LOADING

The cyclic shear strength of joints is an important measure of seismic behavior. Therefore, researchers and designers must pay attention to a reasonable determination of joint shear strength under cyclic loading. Tests have shown that concrete in joints is always seen to deteriorate and joint strength decreases notably with increase in number of load cycles and peak deflection of beams. The same factors affecting monotonic shear strength have been observed to influence joints under cyclic loading.

To obtain a simple and feasible method for predicting cyclic shear strength, the monotonic strength is modified using a statistical analysis of a large number of tests.

5.1 Definition of Cyclic Shear Strength

Cyclic shear strength of joints can be defined as the strength value Q_c corresponding to the load P_c which lies anywhere between the maximum peak load P_m and the worst peak load P_w obtained during some cyclic load history (Fig. 5.1). The curve connecting the peaks in each cycle is referred to as an envelope. The shape of the envelope curve is similar to that of the monotonic curve, but may be a bit lower [34-36,67]. The ultimate strength under monotonic loading and the strength deterioration with cycling are reflected on the envelope curve. It should be noted that under cyclic loading, a specimen may reach the flexural capacity of the beams but suffer strength degradation by shear failure of the joint.

5.2 Influence of Cyclic Loading

Deflections of a reinforced concrete frame under lateral loading are shown in Fig. 5.2. In order to simply and economically investigate the behavior of joints, isolated test specimens, both

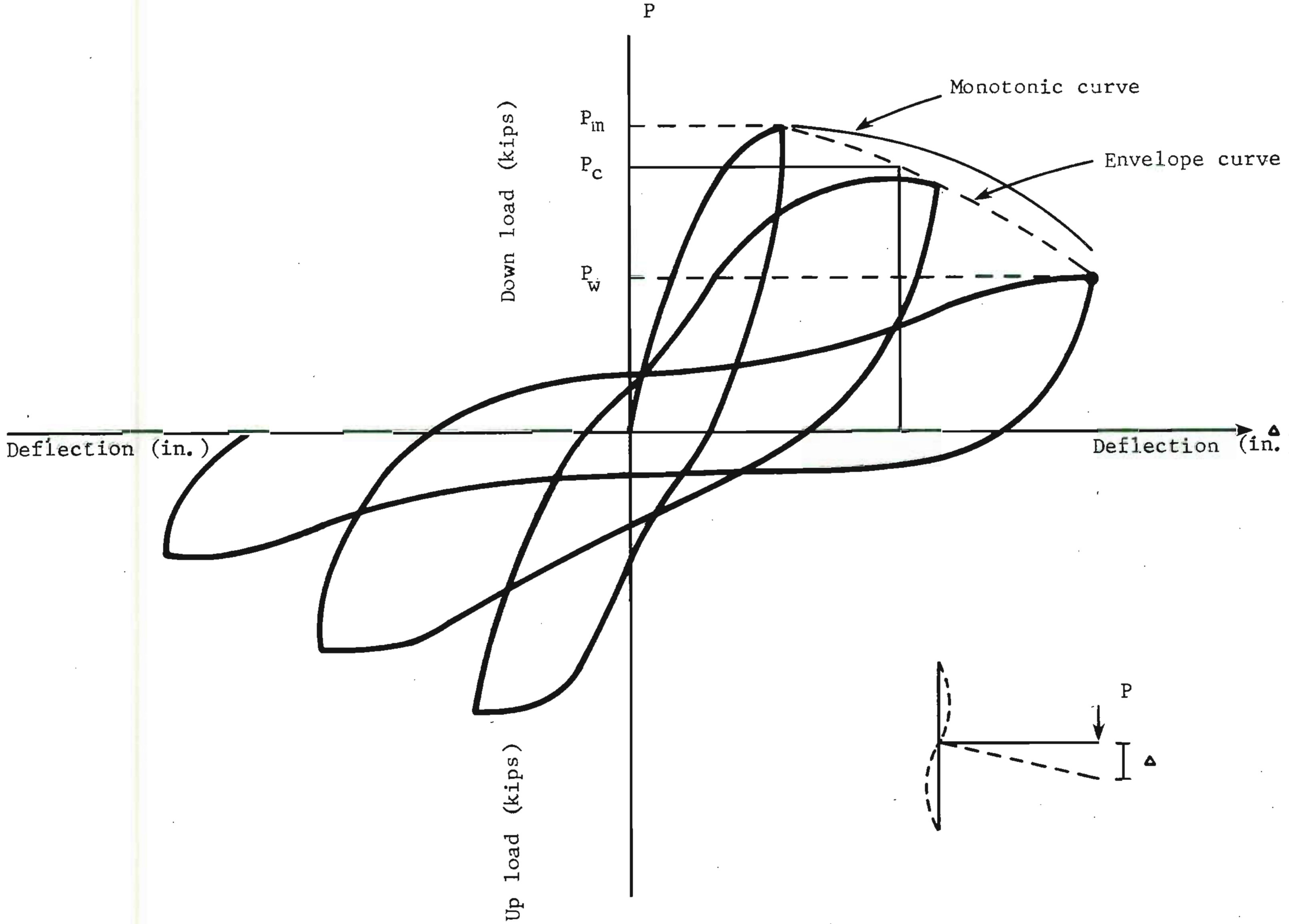


Fig. 5.1 Load-deflection envelope curve under cyclic loading--the worst peak load P_w

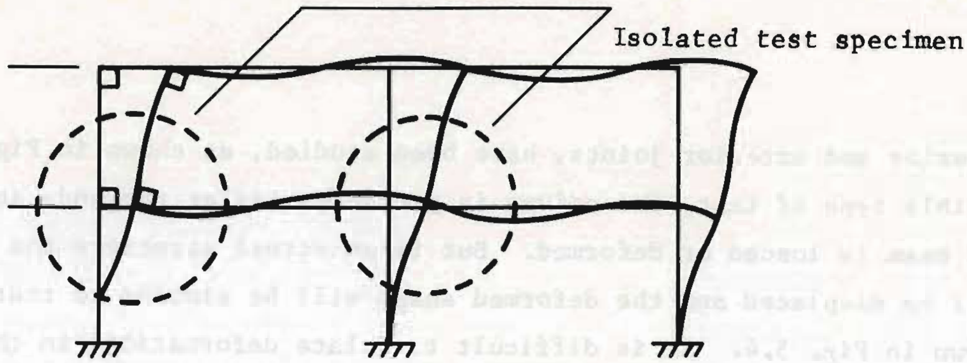


Fig. 5.2 Frame deflection under lateral loads

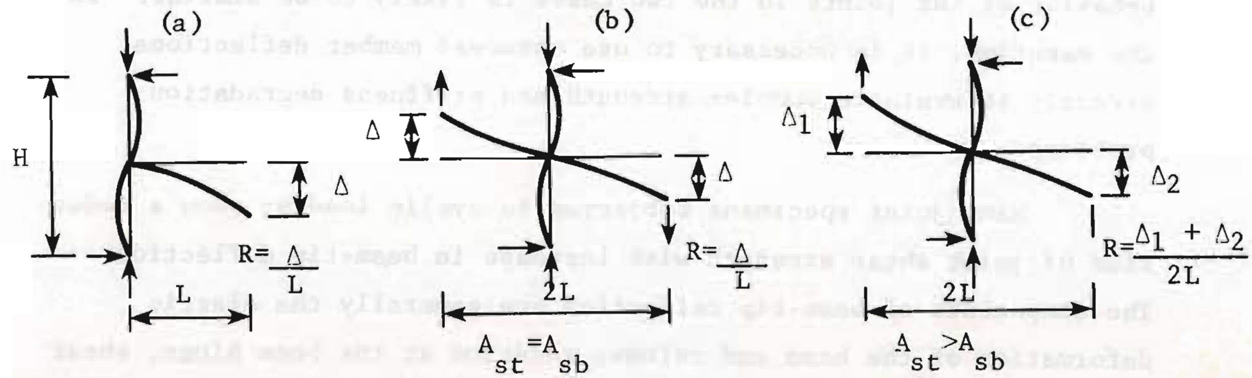
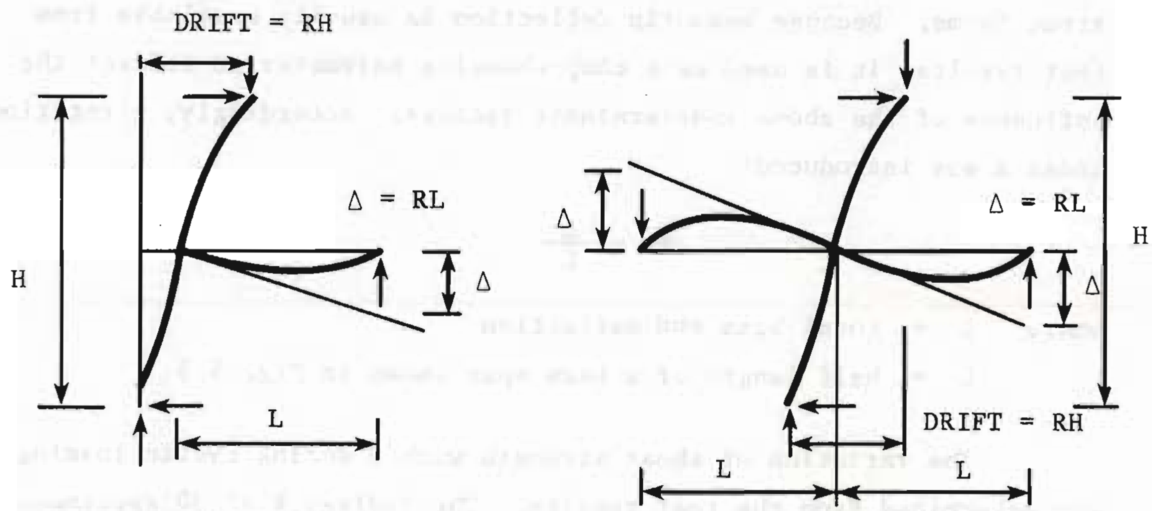


Fig. 5.3 Test joint deformation index R for
(a) exterior joint
(b) interior joint with equal top and bottom bars in beams
(c) interior joint with unequal top and bottom bars in beams



(a) Exterior joint (b) Interior joint

Fig. 5.4 Drift considerations in actual structures interior and exterior joints, have been studied, as shown in Fig. 5.3. In this type of test, the column is pin-supported at the ends and the beam is loaded or deformed. But in an actual structure the column will be displaced and the deformed shape will be similar to that shown in Fig. 5.4. It is difficult to relate deformations in the test specimen to drift values in a real structure. However, if the column is stiff and the beam hinging occurs in a test specimen, the behavior of the joints in the two cases is likely to be similar. In the meantime, it is necessary to use observed member deflections directly to evaluate complex strength and stiffness degradation problems.

Many joint specimens subjected to cyclic loading show a reduction of joint shear strength with increase in beam-tip deflection. The components of beam-tip deflection are generally the elastic deformation of the beam and column, rotation at the beam hinge, shear distortion of the joint, and deterioration of beam reinforcement within the joint. Reduction of panel stiffness and degradation of concrete strength results in a decrease in joint shear strength. The reduction of concrete strength may have a significant influence on shear strength, especially in those joints where the compressive strut forms. Because beam tip deflection is usually available from test results, it is used as a comprehensive parameter to reflect the influence of the above indeterminate factors. Accordingly, a rotation index R was introduced:

$$R = \frac{\Delta}{L}$$

where Δ = total beam end deflection

L = half length of a beam span shown in Fig. 5.3

The variation of shear strength with R during cyclic loading was determined from the test results. The indices R of 30 specimens were found at different control deformations. Based on a statistical analysis of the data (Fig. 5.5), the reduction coefficient η of shear

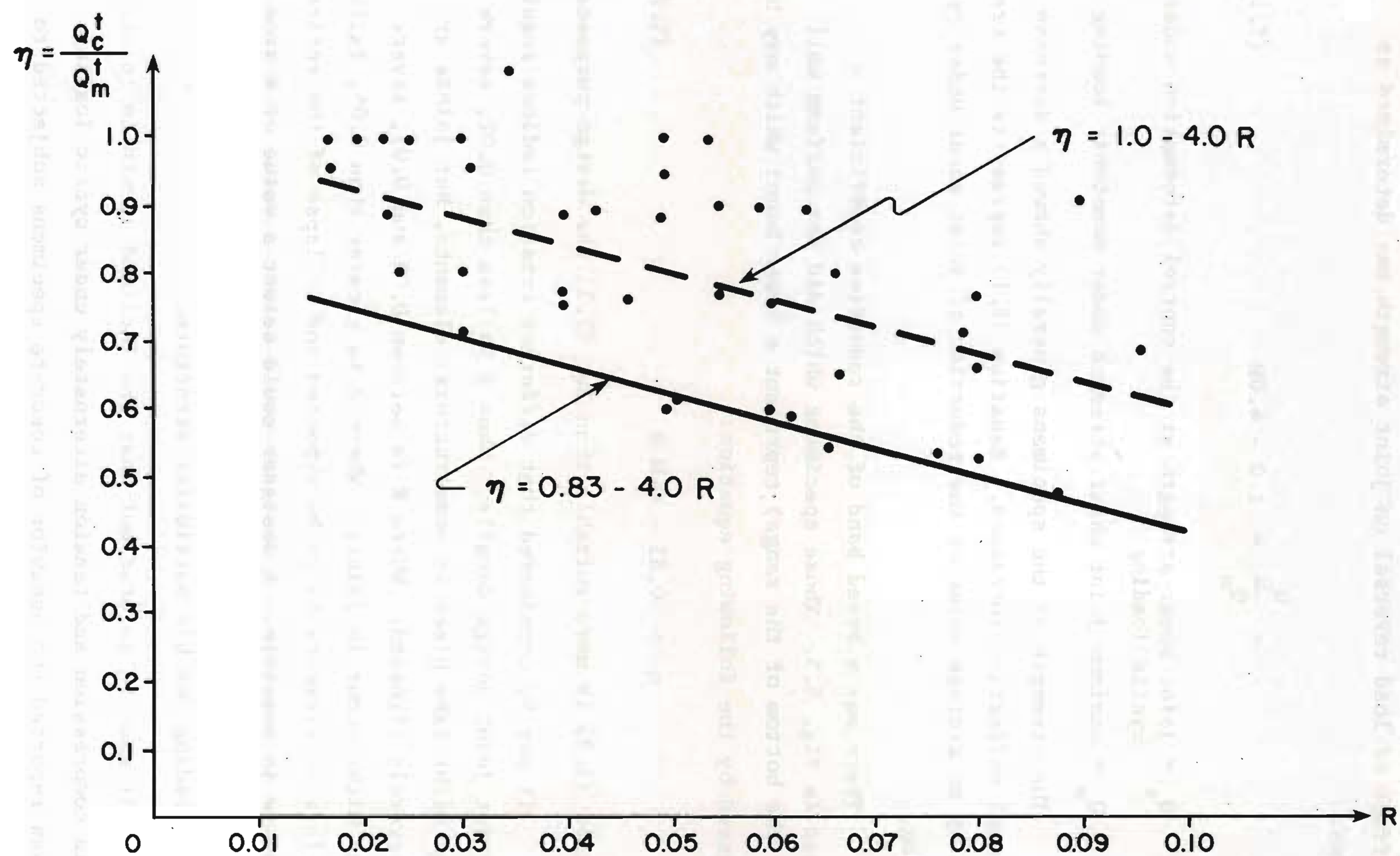


Fig. 5.5 Effect of load reversal on joint strength

strength Q_c/Q_m under cyclic loading versus the index R which reflects the effect of load reversal on joint strength was determined as follows:

$$\eta = \frac{Q_c}{Q_m} = 1.0 - 4.0R \qquad (5.1)$$

where Q_c = joint shear strength at the control deformation under cyclic loading

Q_m = maximum joint shear strength under monotonic loading

The strength of the specimens generally showed a decrease as the total deflection increased. Equation (5.1) represents the trend given by an average value of the reduction of joint shear under cyclic loading.

There was a broad band of the reduction coefficient η plotted in Fig. 5.5. Those specimens which did not perform well (near the bottom of the range) represent a lower bound which may be expressed by the following equation:

$$\eta = 0.83 - 4.0\ R \qquad (5.2)$$

Equation (5.2) is more suitable than Eq. (5.1) for design purposes.

It may be considered that different rotation indices require different joint design details. When R is less than 0.02, severe damage might take place in nonstructural elements, but joints are not severely stressed. Where R is between 0.02 and 0.05, severe damage might occur in joints. Where R is greater than 0.05, failure of joints or elements is to be expected and collapse of the entire structure is possible. A designer could select a value of R from Fig. 5.5 in accordance with the requirement of joint strength under cyclic loading for his particular structure.

It must be pointed out that the inclined struts in joints are in compression and tension alternately under cyclic loading. No one has reported the behavior of concrete specimens subjected to such forces to date. Some idea of the response may be obtained from the hysteretic behavior of concrete under uniaxial compressive cyclic loading [61]. The extent of the strength decay in joints may be more serious than the specimens studied in Ref. 61.

5.3 Determination of Cyclic Shear Strength

Under cyclic reversed loads on the joint, the shear strength gradually decreased due to exhaustion of the compressive concrete strength in the inclined direction. This influence may be expressed by coefficient η. The same variables that determine the monotonic shear strength can be used to develop cyclic strength equations. Using Eqs. (4.13) and (4.14), the modification to monotonic strength is as follows:

For joints with transverse steel and lateral beams where no beam hinge occurs:

$$Q_c = \eta K \zeta \gamma f'_c b_c \sqrt{a_c^2 + a_b^2} \cos \alpha \qquad (5.3)$$

For joints with transverse steel and lateral beams where a beam hinge occurs:

$$Q_c = \eta K \zeta \gamma f'_c b_c a_c \cos \alpha \qquad (5.4)$$

The above equations for predicting cyclic shear strength may be used not only for the failure stage of the joint but any stage of the loading history provided designers determine a desirable value of η that is based on drift limits for a structure.

Equations (5.3) and (5.4) may be used for both interior and exterior joints under cyclic loading.

5.4 Comparison with Test Results

Ninety-six specimens tested in the U.S., Japan, New Zealand, and Canada, for which sufficient data were available to determine that there was strength degradation with cycling, are tabulated in Appendix A. In calculating cyclic shear strength, η was determined using R values based on the reported load-deflection relationships. Values of the ratio of test to calculated shear strength are tabulated for specimens failing first in flexure but degrading in shear and separately for those specimens in which critical failure occurred in the joint. The average and the standard deviation for all 96 specimens taken as a group are 1.090 and 0.340, respectively, in Table 5.1.

TABLE 5.1 CALCULATED VS MEASURED RESULTS--CYCLIC LOADING

Types of Loading	Total No. of Specimens	Joints			Q_{test}/Q_{calc} Average	Std. Dev.
		Interior	Exterior	Subtotal		
Monotonic	164	71	52	123 (F)	1.00	0.23
		36	5	41 (S)	1.10	0.14
Cyclic	96	39	33	72 (F)	1.04	0.35
		19	5	24 (S)	1.22	0.31

Note: (F) - Flexural failure, hinges in beams formed prior to joint failure
(S) - Shear failure, beams did not reach yield

The results indicate that the proposed equations give a reasonable estimate of strength under the cyclic loading, especially where joint shear capacity controls. If the value of the beam-tip deflection corresponding to compression strut failure in joints was known so that the index R could be calculated and the strength reduction coefficient η determined, lower values of average and standard deviation might be obtained. However, considering the variety of sources from which the data base was accumulated, the comparison of test to calculated cyclic shear strength indicates that the compression strut represents an acceptable approach to joint shear calculations.

CHAPTER 6

DESIGN RECOMMENDATIONS FOR SHEAR IN JOINTS

6.1 General Design Criteria

The criteria for satisfactory performance of joints in reinforced concrete frame structures may be summarized as follows:

(1) The total ultimate moment capacity of the column should be greater than the total ultimate moment capacity of the beams along the principal planes at that connection [51,54]. This condition is necessary if column hinges are to be avoided.

(2) Where beam hinges occur at the face of the column, the joint must be designed to transfer forces associated with member hinging in order to guarantee against shear failure [64]. The capacity of the structure should not be jeopardized by possible strength degradation within the joint.

(3) During severe repetition or reversals of deformation, shear degradation at the joint should be prevented. During moderate cyclic deformation, limited cracking may be tolerated but the joint should not exhibit severe distress.

(4) Joint reinforcement necessary to ensure satisfactory performance should not cause undue construction difficulties, but should ensure the integrity of the joint through the deformation history expected.

6.2 Determination of Forces Acting on Joints

(1) The interaction of multidirectional forces, including axial loads, bending, torsion, and shear which the members transfer to the joint as a consequence of the effects of external design loads must be considered.

(2) The forces in the flexural reinforcement at the interface between a member and the joint must be determined as a function of the deformation to which the reinforcement will be subjected. If large deformations are anticipated, stresses in excess of yield should be utilized [51]. For nonseismic loading, stresses equal to yield can be used and for seismic loading stresses 25% greater than yield may be anticipated.

(3) Forces on the joint are determined from a free body of the joint with forces from the members imposed on the joint boundaries.

(4) Forces on the boundaries combine to form an inclined compression strut which transmits the forces through the joint.

6.3 Key Features of Strut Approach

Using test results, the following points were established in developing the strut approach for joint shear strength under monotonic or cyclic loading.

(a) A compressive strut develops early in the load history and is maintained until ultimate load is reached.

(b) The value of compressive stress in the strut may reach the value of concrete compressive strength f'_c under monotonic loading.

(c) Where beam bars lose bond within a joint, the strut has the same compressive-carrying capacity as the strut generated in a joint with good bond conditions.

(d) The same geometric factors influence joint shear strength under both monotonic and cyclic loading.

(e) Loading history has a strong influence on the behavior of the joint. The strength of the strut must be related to the level of deformation and number of repetitions to which the joint will be subjected.

6.4 Simplification of Shear Strength Equation for Design

Equations (5.3) and (5.4) were derived from a large number of tests and were based on defining two geometric parameters, the inclination and width of the compression strut. As indicated in Chapter 4, the inclination of the strut is a function of the aspect ratio of the joint expressed in terms of h_b and h_c, the depth of the beam and column at the joint and of the forces on the joint. In order to simplify the calculation of the geometric parameters, various combinations of axial load and moment, beam and column dimensions, and ratios of beam to column strength were considered. The results of this study are contained in Appendix B. From the material in Appendix B, the basic equation can be written as follows:

$$v_u = \frac{Q_c}{b_c h_c} = \eta\zeta\gamma(1.2 - 0.1f'_c)f'_c\ \lambda \qquad (6.1)$$

For joints when beam hinging may occur at the column face

$$\lambda = \frac{a_c}{h_c} \cos\alpha$$

and

$$\alpha = \tan^{-1} \frac{h_b}{h_c - 2/3a_c} \qquad \text{(inclination of strut)}$$

and for joints where beam hinges cannot form at the column face

$$\lambda = \frac{\sqrt{a_c^2 + a_b^2}}{h_c} \cos\alpha$$

and

$$\alpha = \tan^{-1} \frac{h_b - 2/3a_b}{h_c - 2/3a_c}$$

For both situations:

a_b = depth of compression zone of beam at face of column, in. (Fig. 4.3)

a_c = depth of compression zone of column at joint face, in. (Figs. 4.3 and 4.4)

In lieu of more precise calculations for a_c, a_b, and α, the value of λ may be taken as

$$\beta \Big/ \sqrt{1 + 4\left(\frac{h_b}{h_c}\right)^2}$$

and β has the following values:

Hinging in beams at joint		Hinging prevented at joint	
$M/P \geq 0.3h_c$	$M/P < 0.3h_c$	$M/P \geq 0.3h_c$	$M/P < 0.3h_c$
0.45	0.75	0.65	0.95

For determining β, M and P are the axial load and moment acting on the column when critical joint shear is computed (see Fig. 6.1).

The remaining variables in Eq. (6.1) can be taken as follows:

$\zeta = 0.95 + 4.5\ \rho_s$

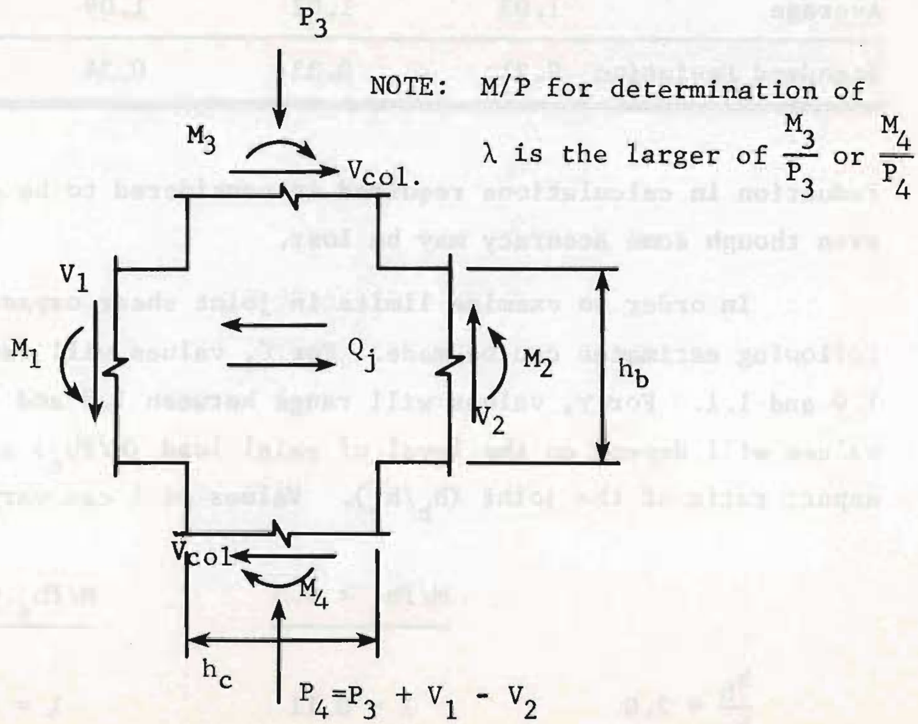
where ρ_s is the volumetric ratio of joint reinforcement and in most cases will be around 0.01. Therefore, ζ may be taken as 1.0 for design purposes.

$\gamma = 0.85 + 0.3\ \frac{W_L}{h_c}$ for lateral beam widths greater than one-half the column width, h_c. If the lateral beam width is equal to the column width, $\gamma = 1.15$. For design purposes, γ could be conservatively taken as 1.0.

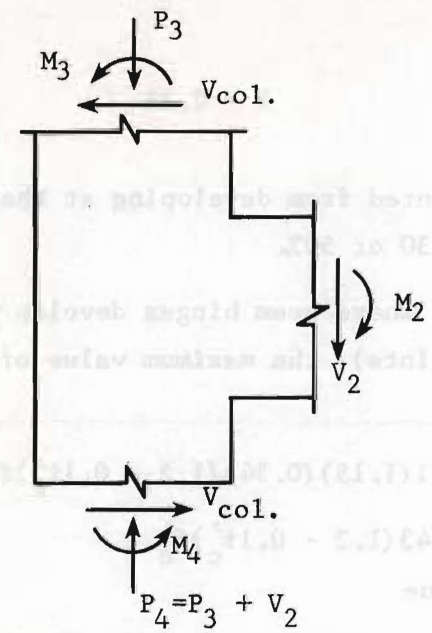
$\eta = 0.83 - 4.0\ \frac{\Delta}{L}$

where Δ is the story deflection and L is the story height. For story drifts (Δ/L) of 2 to 3%, η can be taken as 0.75 to reflect the effect of cyclic or seismic deformations on joint performance.

The average of the ratio of test to calculated values for shear strength are shown in Table 6.1. Note that with the simplifications, the standard deviations shown in Table 6.1 are somewhat higher than indicated in Tables 4.1 and 5.1, but are still acceptable. The



(a) Interior joint



(b) Exterior joint

Fig. 6.1 Critical moment and axial load in joint

TABLE 6.1 COMPARISON OF CALCULATION ACCURACY

Equations	Monotonic Loading		Cyclic Loading	
	Original	Simplified	Original	Simplified
Average	1.03	1.02	1.09	1.07
Standard Deviation	0.21	0.33	0.34	0.43

reduction in calculations required is considered to be desirable even though some accuracy may be lost.

In order to examine limits in joint shear capacity, the following estimates can be made. For ζ, values will range between 1.0 and 1.1. For γ, values will range between 1.0 and 1.15. For λ, values will depend on the level of axial load (M/Ph_c) and on the aspect ratio of the joint (h_b/h_c). Values of λ can vary as follows:

	$M/Ph_c \geq 0.3$	$M/Ph_c < 0.3$
$\frac{h_b}{h_c} = 2.0$	$\lambda = 0.11$	$\lambda = 0.18$
$\frac{h_b}{h_c} = 1.0$	$\lambda = 0.20$	$\lambda = 0.34$
$\frac{h_b}{h_c} = 0.5$	$\lambda = 0.32$	$\lambda = 0.53$

If hinges are prevented from developing at the joint, the value of λ may be increased 30 or 50%.

For a joint where beam hinges develop with $h_b/h_c = 1.0$ (typical of many joints), the maximum value of joint shear can be taken as

$$v_{u\,max} = 1.1(1.15)(0.34)(1.2 - 0.1f'_c)f'_c$$

$$= 0.43(1.2 - 0.1f'_c)f'_c \qquad (6.2)$$

and the minimum value

$$v_{u\,min} = 1.0(1.0)(0.20)(1.2 - 0.1f'_c)f'_c$$

$$= 0.2(1.2 - 0.1f'_c)f'_c$$

Where beam hinges are prevented from forming at the column face,

$$v_{u\,max} = \frac{0.95}{0.75}\,(0.43(1.2 - 0.1f'_c)f'_c$$

$$= 0.54(1.2 - 0.1f'_c)f'_c \qquad (6.3)$$

and $$v_{u\,min} = \frac{0.65}{0.45}\,(0.20)(1.2 - 0.1f'_c)f'_c = 0.29(1.2 - 0.1f'_c)f'_c$$

The values of maximum and minimum joint shear are plotted in Fig. 6.2. Note that the curves tend to bracket $v_u = 20\sqrt{f'_c}$ as proposed by Meinheit [11]. It should also be noted that the higher strength of joints in situations where beam hinging at the joint is prevented has been reported by Paulay, et al [22] and incorporated into some design specifications [52].

For joints in structures subjected to large cyclic deformations, the curves are reduced by a factor of 0.75 and are shown in Fig. 6.3.

6.5 Joint Shear Capacity--Design Specification Format

For joints with $2/3 < h_b/h_c < 3/2$, the shear strength of the joint connecting members in frame systems shall be taken as

$$v_u = \frac{Q_c}{\phi b_c h_c} = \gamma\lambda(1.2 - 0.1f'_c)f'_c \qquad (6.4)$$

where Q_c is the shear on a plane through the joint, and b_c is the column width and h_c is the column depth in the direction of applied shear. If the beam width is less than 3/4 the column width b_c, the shear strength shall be calculated by replacing b_c by $b_b + b_c/2$. ϕ is the understrength factor for joint shear (see Table 6.2).

The value of λ shall be taken as

$$\lambda = \frac{\beta}{\sqrt{1 + 4(h_b/h_c)^2}} \qquad (6.5)$$

where h_b and h_c are the depth of the beam and column, respectively, and β values are shown in Table 6.2.

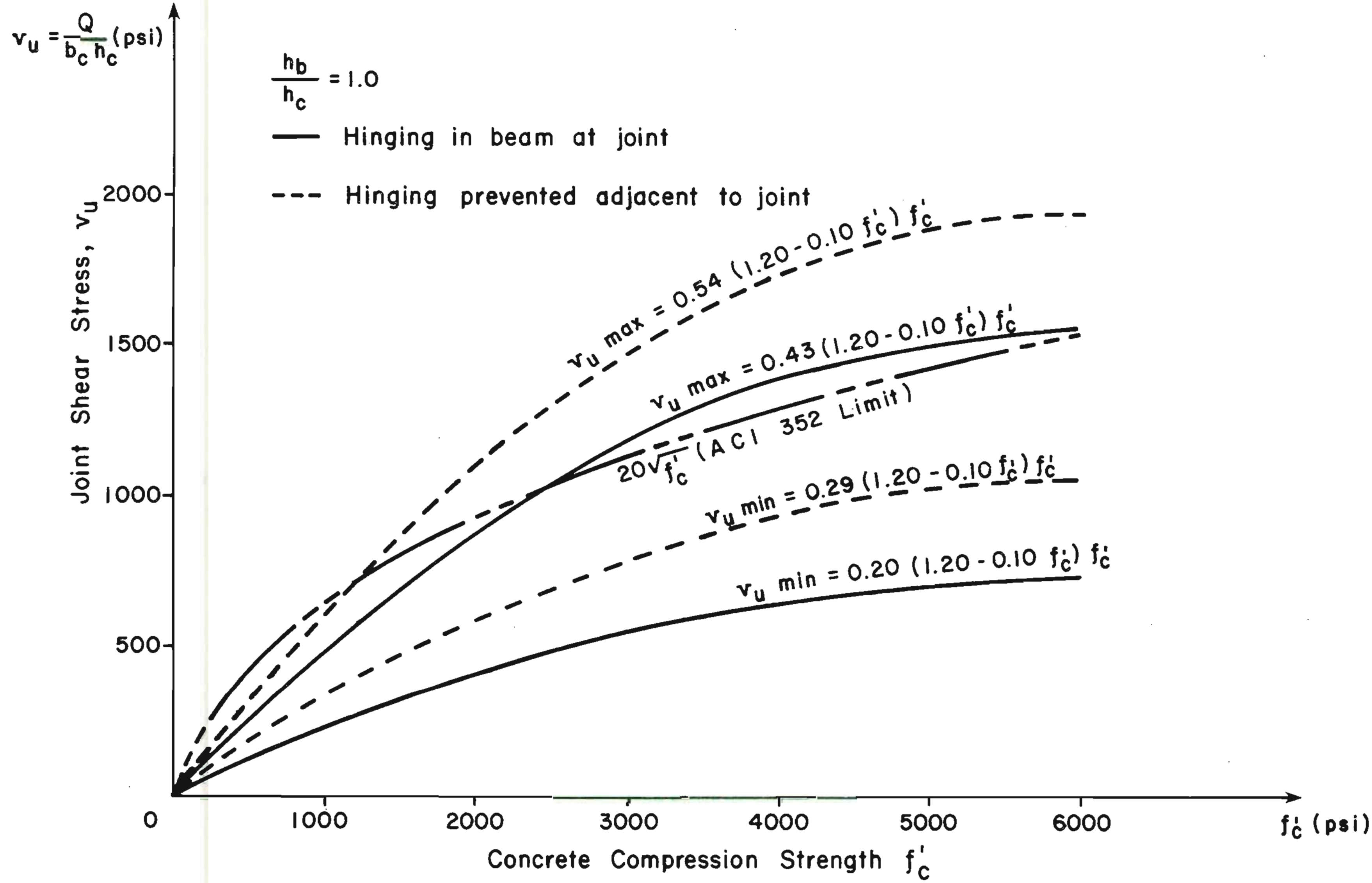


Fig. 6.2 Joint shear strength under nonseismic loading

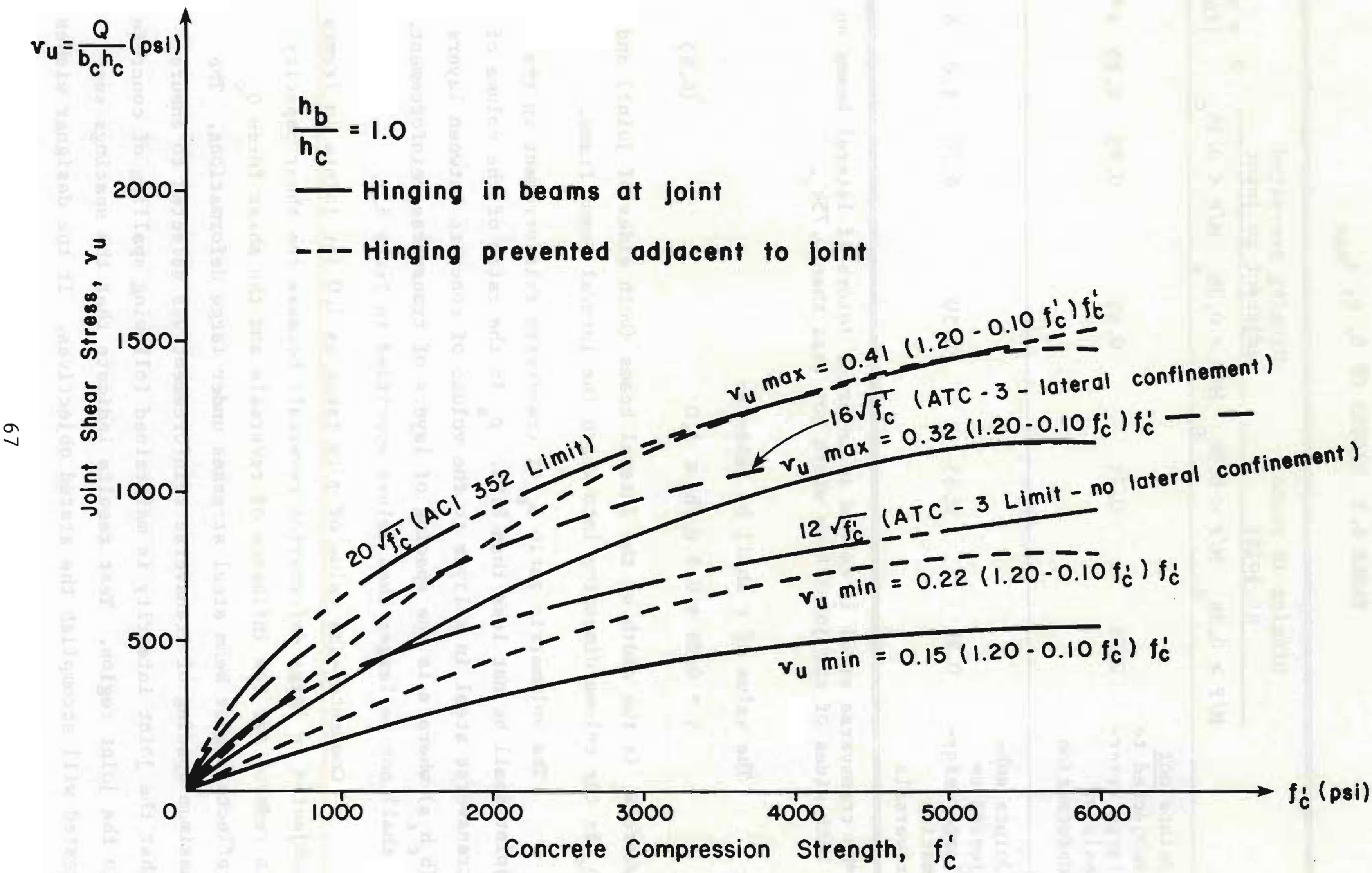


Fig. 6.3 Range of joint shear strength under seismic loading

TABLE 6.2 VALUES OF β, Q, s_{max}

	Hinging in beams at joint		Hinging prevented adjacent to joint		ϕ	s max. (in.)
	$M/P \geq 0.3h_c$	$M/P < 0.3h_c$	$M/P \geq 0.3h_c$	$M/P < 0.3h_c$		
Joints not subjected to large reversals of deformation	0.45	0.75	0.65	0.95	0.85	6*
Joints subjected to large deformation reversals	0.35	0.55	0.50	0.70	1.0	4

*No transverse steel is needed in interior joints if lateral beams on both sides of the joint have width not less than $0.75h_c$.

The value of γ shall be taken as

$$\gamma = 0.85 + 0.3\ W_L/h_c \leq 1.0 \qquad (6.6)$$

where W_L is the width of the lateral beams (both sides of joint) and h_c is the column dimension into which the lateral beams frame.

The volumetric ratio ρ_s of transverse reinforcement in the joint shall be not less than 0.01. ρ_s is the ratio of the volume of transverse steel in a layer to the volume of concrete between layers ($b_c h_c s$) where s is the spacing of layers of transverse reinforcement. s shall not be larger than values specified in Table 6.2.

Comments--The value of ϕ is taken as 1.0 for joints in frames subjected to large deformation reversals because the shear capacity is reduced for the influence of reversals and the shear force Q_c reflects higher beam steel stresses under large deformations. The maximum spacing of transverse reinforcement was selected to ensure that the joint integrity is maintained following spalling of concrete in the joint region. Test results indicate that the spacings suggested will accomplish the stated objectives. If the designer wishes to include directly the influence of transverse reinforcement, an additional factor $\zeta = 0.95 + 4.5\ \rho_s$ may be included where large values of ρ_s are considered.

Design Examples--Several design examples are shown in Appendix C to illustrate the use of the compression strut mechanism as reflected in the design specifications of this section.

6.6 Comparison with Other Codes

Figures 6.2 and 6.3 show a comparison of upper bound for joint shear strength under nonseismic loading and seismic loading obtained from different codes. This method covers a band for joint shear stress with the upper bound $[v_{u\ max} = 0.54(1.20 - 0.10f'_c)f'_c]$ and the lower bound $[v_{u\ min} = 0.20(1.20 - 0.10f'_c)f'_c]$ under nonseismic loading (Fig. 6.2). The upper bound for joint shear strength $[v_{u\ max} = 0.41(1.20 - 0.10f'_c)f'_c]$ and lower bound $[v_{u\ min} = 0.15(1.20 - 1.10f'_c)f'_c]$ under seismic loading are shown in Fig. 6.3. The limits in ACI 352 ($20\sqrt{f'_c}$) and ATC-3 ($16\sqrt{f'_c}$) are both lower than that derived herein (Fig. 6.2). Using $20\sqrt{f'_c}$ for nonseismic loading may be a reasonable average value, but large deviations are to be expected.

Figures 6.4, 6.5, 6.6, and 6.7 show a comparison of shear strength v_u of joints under different conditions using sample joints. In general, the shear stress obtained from several codes lies in a fairly narrow range. The approach outlined in Section 6.5 indicates that shear capacity of joints is not influenced significantly by transverse reinforcement. The approaches outlined in the report of ACI 352 and the New Zealand codes are highly dependent on transverse reinforcement under seismic loading. The ultimate shear stresses of $12\sqrt{f'_c}$ for a joint in a seismic zone given by ATC-3 and Meinheit fall within the band obtained from this method (Fig. 6.8). Again, this method which is based on a large number of test data, demonstrates that the suggestions of ATC-3 and Meinheit are acceptable. The band of joint shear stress derived from the strut approach brackets shear

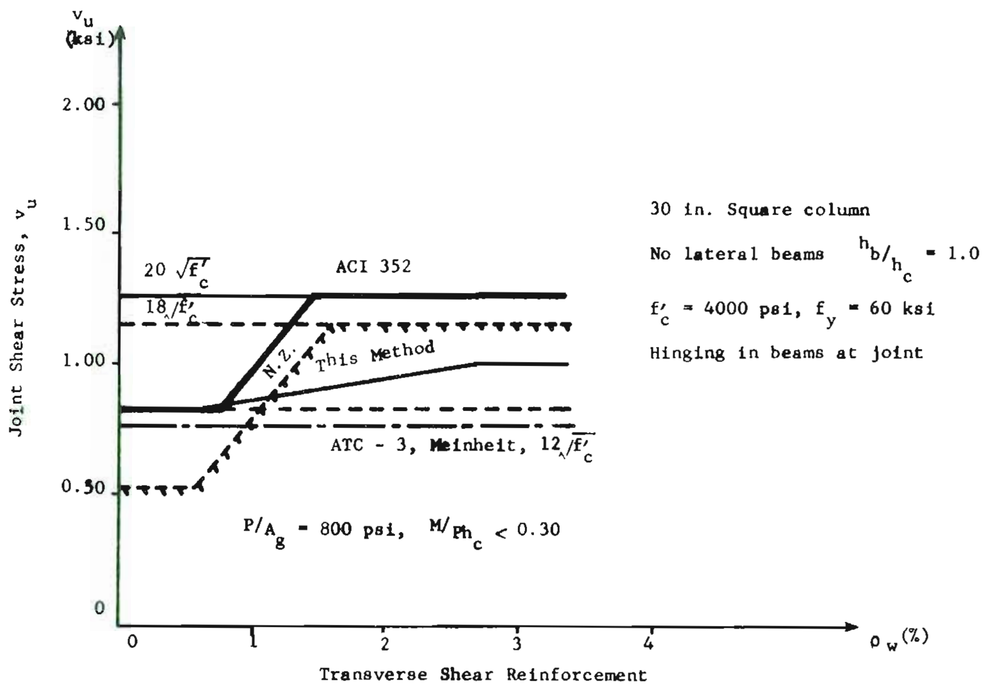


Fig. 6.4 Comparison of joint shear capacity, seismic loading, axial load

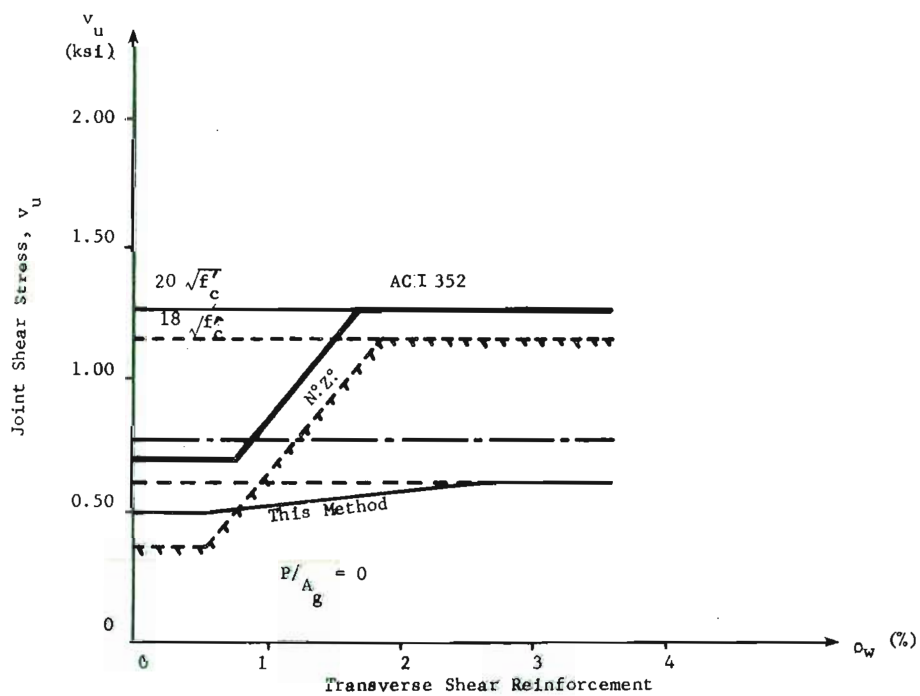


Fig. 6.5 Comparison of joint shear capacity, seismic loading, no axial load

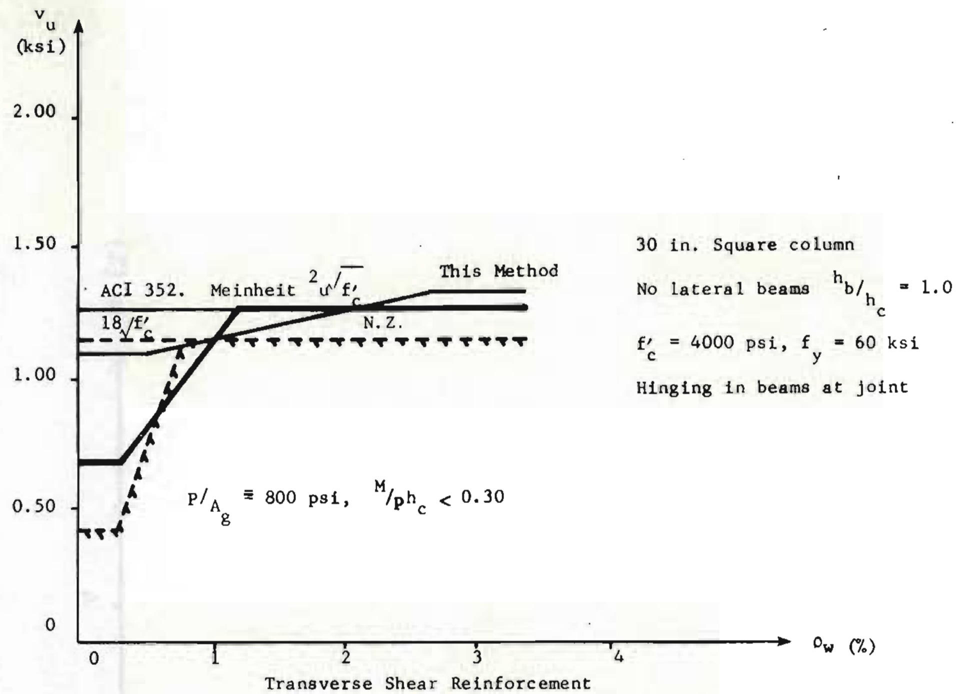


Fig. 6.6 Comparison of joint shear capacity, monotonic loading, axial load

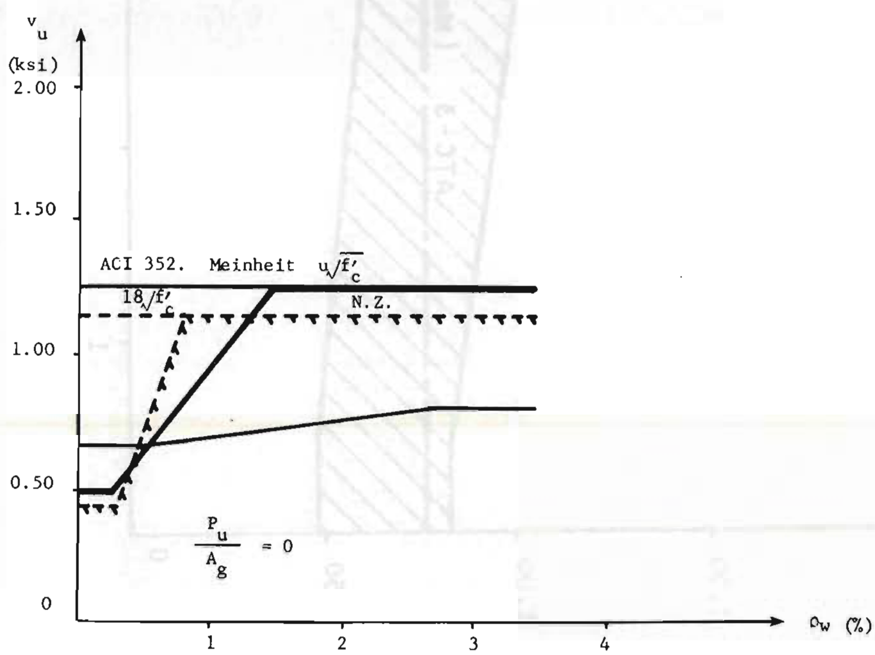


Fig. 6.7 Comparison of joint shear capacity, monotonic loading, no axial load

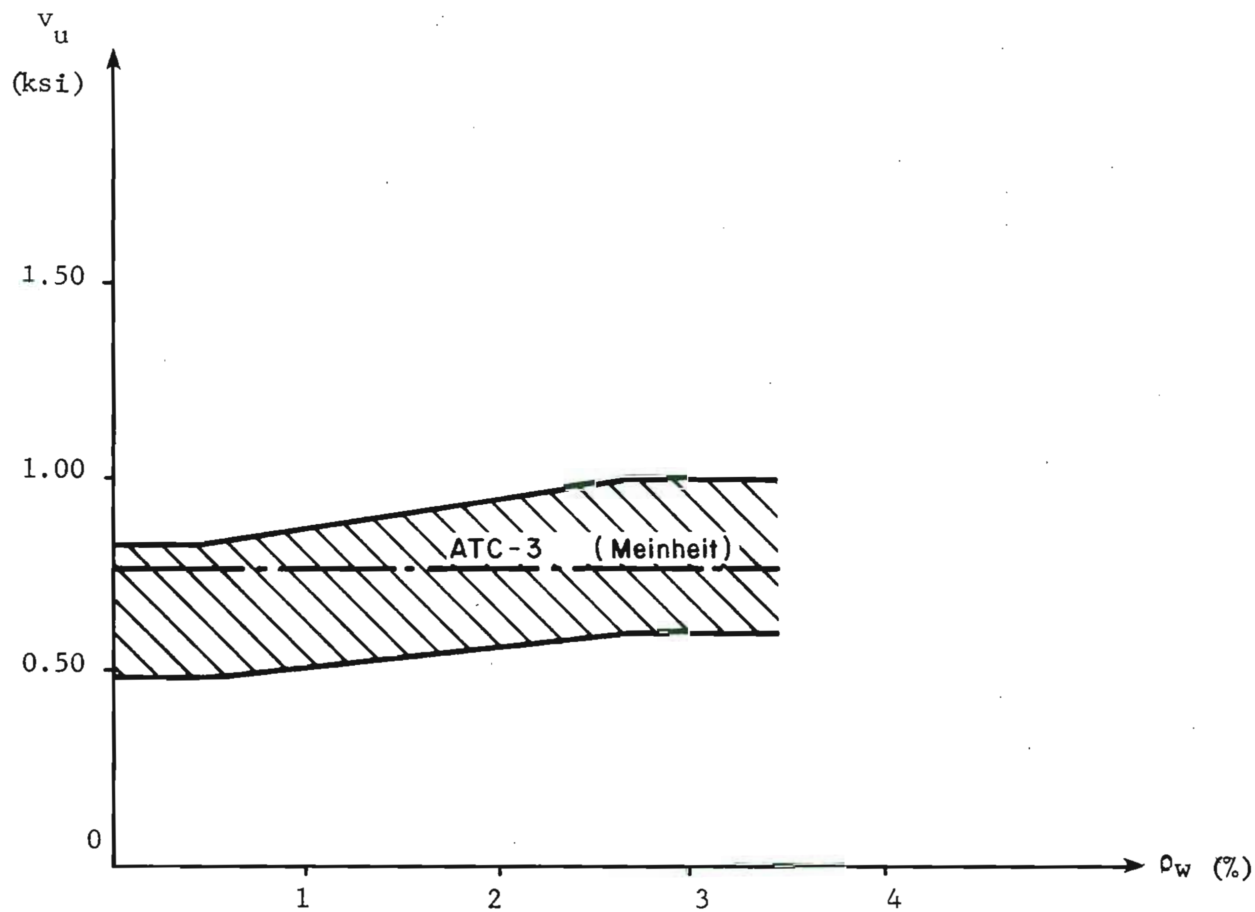


Fig. 6. 8 Comparison of strut approach with ATC-3 equations, seismic loading

stresses determined using the ACI 352 method for nonseismic loading (see Fig. 6.9).

6.7 Joint Details for Shear Reinforcing

Although the volume of transverse reinforcement does not have a substantial influence on shear strength, the arrangement and amount of joint reinforcement does influence the joint performance, especially the stiffness. Joint stiffness was not studied in this report. Therefore, joint details should not be overlooked in design. The following suggestions may be adopted if construction conditions permit:

(a) Closing hoops with hooks at the ends of not less than 135° and ten bar diameter extensions. Joint hoops should be #3 bar minimum. The maximum spacing of ties in nonseismic zones is 6 in. and 4 in. in seismic zones.

(b) Providing additional confining bars at right angles to the shear reinforcement, as shown in Fig. 6.10(a).

(c) Putting a closely knit cage both in longitudinal and lateral directions, as shown in Fig. 6.10(b).

(d) Laying down some layers of welded mesh in joints as shown in Fig. 6.10(c).

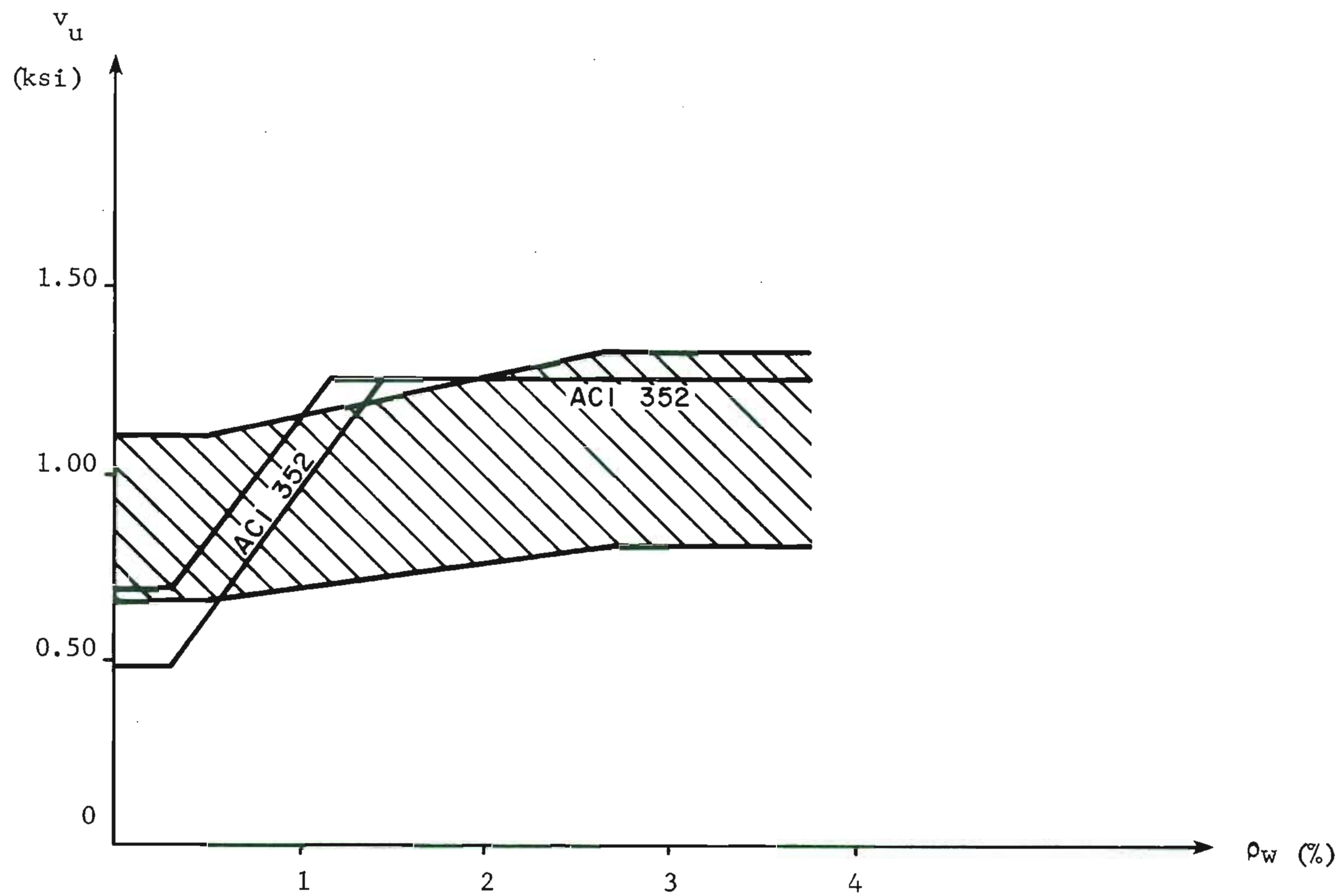


Fig. 6.9 Comparison of strut method and ACI 352, nonseismic loading

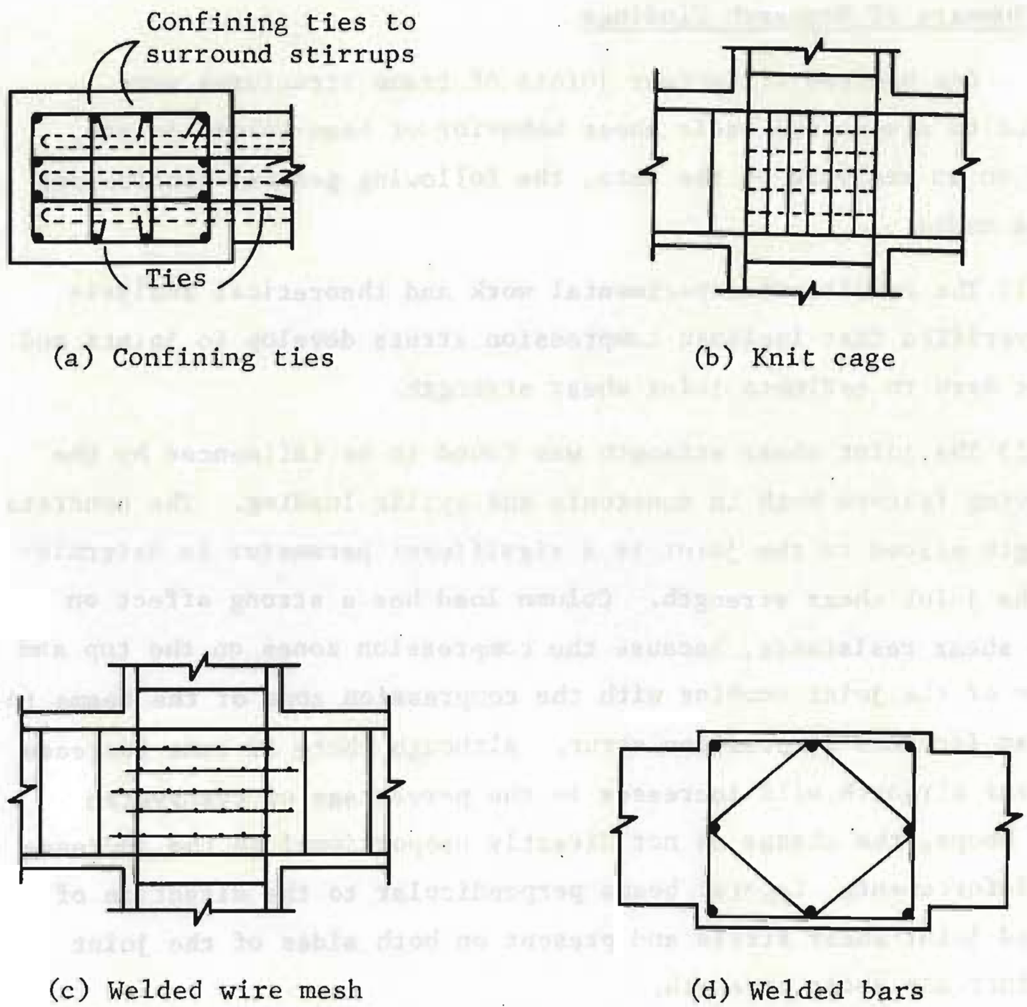


Fig. 6.10 Possible arrangement of joint reinforcement

CHAPTER 7

SUMMARY

7.1 Summary of Research Findings

One hundred sixty-four joints of frame structures were studied to assess the basic shear behavior of beam-column joints. Based on an analysis of the data, the following general conclusions may be made:

(1) The results of experimental work and theoretical analysis have verified that inclined compression struts develop in joints and may be used to estimate joint shear strength.

(2) The joint shear strength was found to be influenced by the following factors both in monotonic and cyclic loading. The concrete strength placed in the joint is a significant parameter in determining the joint shear strength. Column load has a strong effect on joint shear resistance, because the compression zones on the top and bottom of the joint combine with the compression zone of the beams to form an inclined compression strut. Although there is some increase in shear strength with increases in the percentage of transverse joint hoops, the change is not directly proportional to the increase in reinforcement. Lateral beams perpendicular to the direction of applied joint shear stress and present on both sides of the joint also increase shear strength.

Therefore, the main factors which should be taken into account in joint shear strength calculations both for monotonic and cyclic loading are: concrete strength, geometry, column load, transverse reinforcement, and lateral beams.

(3) The character of cyclic loading is an important factor for joints under cyclic loading. Cyclic loading can cause a reduction

of joint shear strength. The reduction is related to the amount of story drift relative to story height.

(4) Comparing the shear strength estimated using the compression strut with the results of 164 specimens, the strut approach appears to be both simple and reasonably accurate. The approach is easy to apply to design of both interior and exterior joints under seismic or nonseismic loading.

7.2 Suggestions for Future Research

Although this research identified the formation of compression struts in joints and principles for predicting joint shear strength using a strut mechanism were developed in the form of design recommendations, many unanswered questions remain regarding shear behavior of joints. In this light, research on the following topics may be useful.

(1) Strength of concrete in struts under cyclic loading

A reduced strength of concrete should be used in evaluating joint shear strength due to the strength decay under cyclic loading producing a set of intersecting crack patterns. Concrete in the joint is subjected to uniaxial compressive cyclic loading alternately from the two inclined intersecting struts. Information is needed to define the performance of concrete under such conditions.

(2) Influence of size of members framing into joints

When the ratio of b_b to b_c is less than 0.75, the value of $(b_b + b_c)/2$ can be taken for b_c. The reason is that narrow beams are unlikely to generate a strut with a width equal to the column width b_c. More tests are needed to evaluate the effect of the ratio of b_b to b_c. Research is also needed to estimate the performance of the joint where the beam is wider than the column.

(3) Minimum and maximum shear reinforcement in joints

Though minimum transverse steel is governed by confinement requirements in all current codes and maximum transverse steel is given by limits on joint shear capacity, more data are needed to explain more accurately the role of reinforcement in carrying joint shear forces.

(4) Value of story drift used in design

The reduction in shear strength under cyclic loading is related to the deformations to which the structure is subjected. Drift for design should be related to overall structural response at the time failure occurs. However, no data are available on the difference between a test specimen and a real assembly in a structure in terms of specimen deflection and story drift.

(5) Anchorage of straight bars through joints

As there is a loss of bond through the joint produced by the adverse effects of loading reversals, beam bars which should be in compression are in tension. Bar slip has been observed in tests. However, bond-slip relations are not clearly defined, although some mathematical models have been presented. Accordingly, studies are needed to evaluate bond-slip relations as well as the conditions producing rapid deterioration of the anchorage capacity of the bar.

(6) Influence of axial column loads

The equation presented in this report for joint shear strength indicates that compressive axial loads are beneficial. Very little work has been done on the behavior of members, especially columns, under tensile loading or under variable axial loadings ranging from tension to compression.

(7) Stiffness of joints

Additional work is needed to correlate the compression strut with determination of joint stiffness under both monotonic and cyclic loading.

A P P E N D I C E S

A P P E N D I X A

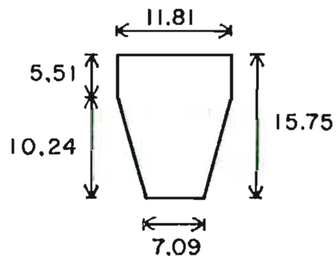
TEST RESULTS AND COMPARISON OF CALCULATED JOINT SHEAR STRENGTHS WITH MEASURED DATA

KEY TO TABLE

Col. 0	Specimen identification (Reference)
Cols. 1-3	Column dimensions (see Fig. A.1), in.
Cols. 4-7	Column reinforcement per layer (see Fig. A.1), in. Note that if there are corner bars only, entries will be in A_{s1} and A_{s2} and will be equal.
Col. 8-9	Column material properties, ksi
Col. 10-12	Beam dimensions (see Fig. A.1), in.
Col. 13-14	Beam reinforcement (see Fig. A.1), $in.^2$
Col. 15-16	Beam material properties, ksi
Col. 17	Joint reinforcement, no. of tie legs per layer, size, and spacing of layers
Col. 18	Volumetric ratio of joint reinforcement, %
Col. 19-20	Joint material properties, ksi
Col. 21-22	Lateral beam dimensions (see Fig. A.1), in. In Col. 22, a superscript 1 indicates a lateral beam on one side only, a 2 indicates lateral beams on both sides.
Col. 23	Column load, kips
Col. 24	Type of Joint, <u>I</u>nterior or <u>E</u>xterior I* indicates Interior with special anchorage detail E* indicates Exterior with special anchorage detail
Col. 25	Reported failure mode, first cycle <u>F</u>lexural failure in beam <u>S</u>hear failure in joint
Col. 26	Measured maximum joint shear, kips
Col. 27	Calculated joint shear capacity, kips [Eqs. (4.13, (4.14)]
Col. 28	Ratio of test to calculated shear strength
Col. 29	Measured cyclic joint shear, kips
Col. 30	Calculated cyclic joint shear capacity, kips [Eqs. (5.3), (5.4)]
Col. 31	Ratio of test to calculated shear strength

Special Note for Research Group of China (47):

1. The trapezoid shows the beam section Type J.
2. The specimens have cast-in-place columns and precast beams except Type J specimens and KJ-1.



Special Note for U.S.S.R. (45):

1. Steel areas in U.S.S.R. data were evaluated using Code of China.
2. The values of Columns 26-28 (Q_t, Q_c, and Q_t/Q_c) are for compressive forces along inclined strut for Russian data.

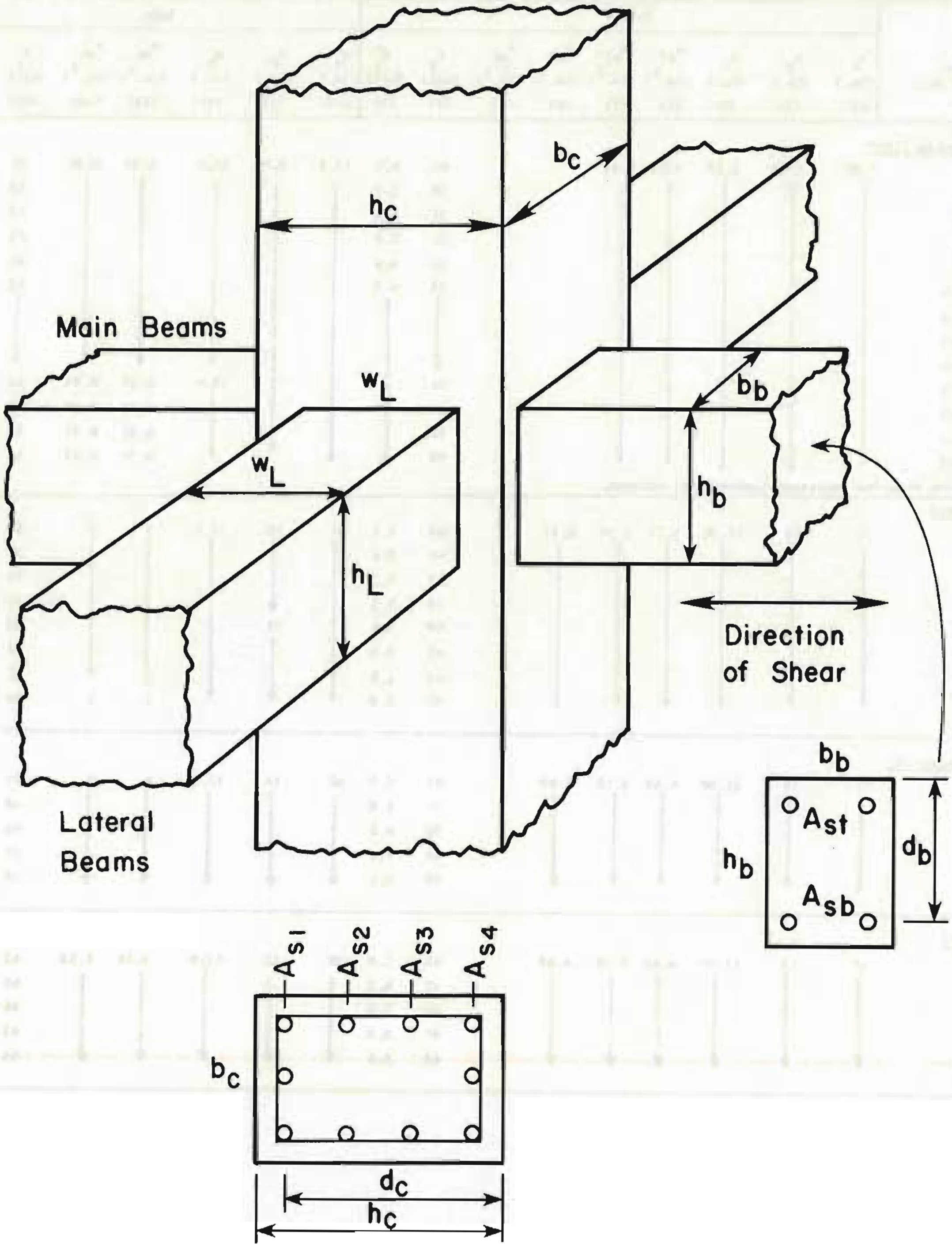


Fig. A.1 Definition of joint geometry

Specimen (Reference No.) (0)	Column h_c (in.) (1)	b_c (in.) (2)	d_c (in.) (3)	A_{s1} (in.^{2}) (4)	A_{s2} (in.^{2}) (5)	A_{s3} (in.^{2}) (6)	A_{s4} (in.^{2}) (7)	f_y (ksi) (8)	f'_c (ksi) (9)	Beam h_b (in.) (10)	b_b (in.) (11)	d_b (in.) (12)	A_{st} (in.^{2}) (13)	A_{sb} (in.^{2}) (14)	f_y (ksi) (15)	f'_c (ksi) (16)
Higashi-Ohwada (31)*																
1SD35Aa-4	7.87	7.87	6.69	0.49	0.49			61	4.4	11.8	5.9	11.0	0.37	0.37	61	4.1
4SD35Aa-7	↓	↓	↓	↓	↓			58	5.5	↓	↓	↓	↓	↓	58	5.5
5SD35Aa-8	↓	↓	↓	↓	↓			58	5.5	↓	↓	↓	↓	↓	58	5.5
8SD35Ba-2	↓	↓	↓	↓	↓			61	4.4	↓	↓	↓	↓	↓	61	4.4
9SD35Ba-3	↓	↓	↓	↓	↓			61	4.6	↓	↓	↓	↓	↓	61	4.6
10LSD35Aa-1	↓	↓	↓	↓	↓			58	6.0	↓	↓	↓	↓	↓	58	6.0
11LSD35Aa-2	↓	↓	↓	↓	↓			↓	↓	↓	↓	↓	↓	↓	↓	↓
12LSD35Ab-1	↓	↓	↓	↓	↓			↓	↓	↓	↓	↓	↓	↓	↓	↓
13LSD35Ab-2	↓	↓	↓	↓	↓			↓	↓	↓	↓	↓	↓	↓	↓	↓
14LSR24Aa-1	↓	↓	↓	↓	↓			44	↓	↓	↓	11.0	0.39	0.39	44	↓
15LSR24Ab-1	↓	↓	↓	↓	↓			44	↓	↓	↓	↓	0.39	0.39	44	↓
16LSD35Ba-1	↓	↓	↓	↓	↓			58	↓	↓	↓	↓	0.37	0.37	58	↓
17LSD35Ba-2	↓	↓	↓	↓	↓			58	↓	↓	↓	↓	0.37	0.37	58	↓
Uzumeri (43)																
SP1	15	15	12.50	2.37	1.58	2.37		48	5.6	20	12	17.6	3	2	50	4.5
SP2	↓	↓	↓	↓	↓	↓		49	5.6	↓	↓	↓	↓	↓	51	4.5
SP3	↓	↓	↓	↓	↓	↓		49	5.7	↓	↓	↓	↓	↓	51	3.9
SP4	↓	↓	↓	↓	↓	↓		48	5.5	↓	↓	↓	↓	↓	51	4.5
SP5	↓	↓	↓	↓	↓	↓		49	5.4	↓	15	↓	↓	↓	50	4.6
SP6	↓	↓	↓	↓	↓	↓		49	5.5	↓	↓	↓	↓	↓	51	5.2
SP7	↓	↓	↓	↓	↓	↓		49	4.5	↓	↓	↓	↓	↓	51	4.5
SP8	↓	↓	↓	↓	↓	↓		57	3.8	↓	↓	↓	4	3	51	3.8
Hanson-Conner (8)																
I	15	15	12.80	4.68	3.12	4.68		67	5.7	20	12	17.9	4	2	52	3.5
IA	↓	↓	↓	↓	↓	↓		70	5.3	↓	↓	↓	↓	↓	48	3.2
II	↓	↓	↓	↓	↓	↓		70	6.0	↓	↓	↓	↓	↓	48	3.7
V	↓	↓	↓	↓	↓	↓		65	5.4	↓	↓	↓	↓	↓	51	3.3
Va	↓	↓	↓	↓	↓	↓		70	5.2	↓	↓	↓	↓	↓	50	5.4
Hanson (9)																
1	15	15	12.80	4.68	3.12	4.68		61	5.6	20	12	17.9	3.16	1.58	63	5.5
2	↓	↓	↓	↓	↓	↓		60	4.9	↓	↓	↓	↓	↓	65	2.9
3	↓	↓	↓	↓	↓	↓		60	5.3	↓	↓	↓	↓	↓	64	5.2
4	↓	↓	↓	↓	↓	↓		61	5.2	↓	↓	↓	↓	↓	63	5.4
5	↓	↓	↓	↓	↓	↓		62	5.4	↓	↓	↓	↓	↓	65	5.2

*An L in the notation indicates lightweight concrete.

Joint				Lateral Beam										
Joint Reinf. (17)	ρ_s (18)	f_y (ksi) (19)	f'_c (ksi) (20)	w_L (in.) (21)	h_L (in.) (22)	Column Load (kips) (23)	Type of Joint (24)	Failure Mode 1st Cycle (25)	Q_t (kips) (26)	Q_c (kips) (27)	Q_t/Q_c (28)	Q_t^* (kips) (29)	Q_c^* (kips) (30)	Q_t^*/Q_c^* (31)
6φ4.1 @ 1.57"	0.8	40	4.4	--	None	17.70	I*	F	36.60	34.20	1.21	--	--	--
↓	↓	↓	5.5	--		17.70	↓	↓	34.43	33.00	1.04	--	--	--
			5.5	--		35.40			36.60	39.90	0.92	--	--	--
			4.3	3.94	11.81[2]	17.70			38.00	29.70	1.28	--	--	--
			4.6	7.87	11.81[2]	17.70			38.30	35.84	1.0[illegible]	--	--	--
			6.0	--	None	17.70			33.01	37.10	0.89	25.37	25.23	1.0
			↓	--	↓	35.40	I		34.90	38.70	0.90	--	--	--
				--		17.70	↓		34.43	36.77	0.94	--	--	--
				--		35.40			33.01	38.70	0.85	--	--	--
				--		17.70	I*		32.40	38.00	0.85	--	--	--
				--		↓	I		32.14	38.00	0.85	32.40	25.84	1.25
				3.94	11.81[2]		I*		34.90	37.10	0.94	34.90	25.23	1.38
				7.87	11.81[2]		I*		34.90	42.60	0.82	--	--	--
None	--	--	4.5	12	20[1]	520	E	F	140.9	180.0	0.78	88	158.4	0.56
None	--	--	4.5	--	None	↓	↓	↓	136.9	182.0	0.75	136.9	166.0	0.82
4-#3 @ 3"	1.2	62	3.9	12	20[1]				145.7	171.0	0.85	79.26	119.0	0.67
4-#4 @ 3"	2.2	55	4.5	12	20[1]				165.0	190.7	0.87	88.58	135.7	0.65
None	--	--	4.6	15	20[1]				136.7	184.4	0.74	136.7	158.6	0.86
8-#4 @ 1.75"	3.8	52	5.3	--	None				163.7	212.8	0.77	163.7	152.4	1.07
4-#4 @ 3"	2.2	53	4.5	--	↓				157.1	189.4	0.83	93.25	133.3	0.70
8-#4 @ 1.75"	3.8	53	3.8	--					188.4	184.2	1.02	124.33	126.0	0.99
5-#4 @ 4.5"	1.4	47	3.5	--	None	644	E	F	178.8	161.5	1.10	160.56	133.7	1.20
5-#3 @ 4.5"	0.8	53	3.2	--	↓	647	↓	↓	168.4	152.1	1.10	99.22	117.3	0.85
5-#4 @ 4.5"	1.4	55	3.7	--		284			169.0	180.0	0.94	169.0	154.8	1.09
None	--	--	3.3	--		636			173.0	155.0	1.11	88.6	123.3	0.72
None	--	--	5.4	12	20[2]	649			173.8	230.0	0.76	172.6	210.6	0.82
6-#4 @ 3.5"	3.5	67	5.5	12	20[1]	640	I	F	255.4	213.4	1.20	230.52	158.8	1.45
5-#4 @ 4.1"	3.0	67	2.9	↓	20[2]	↓	↓	↓	259.9	171.4	1.51	240.30	130.3	1.84
5-#3 @ 4.1"	1.7	74	5.2		20[1]		E		185.9	196.0	0.95	185.9	162.3	1.15
4-#3 @ 6"	1.1	74	5.4	--	None		↓		179.7	193.0	0.93	171.7	169.0	1.02
6-#3 @ 3.5"	1.9	74	5.2	--	None	320			183.6	222.9	0.82	153.0	181.0	0.85

Specimen (Reference No.)	Column									Beam						
	h_c (in.)	b_c (in.)	d_c (in.)	A_{s1} (in.^{2})	A_{s2} (in.^{2})	A_{s3} (in.^{2})	A_{s4} (in.^{2})	f_y (ksi)	f'_c (ksi)	h_b (in.)	b_b (in.)	d_b (in.)	A_{st} (in.^{2})	A_{sb} (in.^{2})	f_y (ksi)	f'_c (ksi)
(0)	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
Hanson-Conner (10)																
7	15	15	12.80	4.68	3.12	4.68		82	5.9	20	12	17.94	4	2	51	5.7
8	↓	↓	↓	↓	↓	↓		82	6.0	↓	↓	↓	↓	↓	44	6.1
9	↓	↓	↓	↓	↓	↓		62	6.0	↓	↓	↓	↓	↓	44	3.8
10	↓	↓	↓	↓	↓	↓		65	5.5	↓	↓	↓	↓	↓	44	5.7
Meinheit-Jirsa (11)																
I	18	13	16	1.75	1.75	1.75	1.75	66	3.8	18	11	16	3.81	2.37	65(top) 59(bot)	3.8
II	18	13	16	2.54	2.54	2.54	2.54	65	6.1	↓	↓	↓	↓	↓	↓	6.1
III	18	13	16	2.82	2.82	2.82	2.82	58	3.9	↓	↓	↓	↓	↓	↓	3.9
IV	13	18	11	4.51	2.26	4.51	--	64	5.2	↓	16	↓	↓	↓	↓	5.2
V	18	13	16	2.54	2.54	2.54	2.54	65	5.2	↓	11	↓	↓	↓	↓	5.2
VI	18	13	15.40	2.54	2.54	2.54	2.54	65	5.3	↓	11	15.40	↓	↓	↓	5.3
VII	13	18	11	4.51	2.26	4.51	--	64	5.4	↓	16	16	↓	↓	↓	5.4
VIII	18	13	15.40	2.54	2.54	2.54	2.54	65	4.8	↓	11	15.40	↓	↓	↓	4.8
IX	18	13	16	2.54	2.54	2.54	2.54	65	4.5	↓	11	16	↓	↓	↓	4.5
X	18	13	16	2.54	2.54	2.54	2.54	65	4.3	↓	11	16	↓	↓	↓	4.3
XI	13	18	11	4.51	2.26	4.51	--	64	3.7	↓	16	16	↓	↓	↓	3.7
XII	18	13	15.40	2.54	2.54	2.54	2.54	65	5.1	↓	11	15.50	↓	↓	↓	5.1
XIII	18	13	16	2.54	2.54	2.54	2.54	65	6.0	↓	11	16	↓	↓	↓	6.0
XIV	13	18	11	4.51	2.26	4.51	--	64	4.8	↓	16	16	↓	↓	↓	4.8
Megget (16)																
M1	15	13	12.56	1.20	1.20			44	4.1	18	10	15.44	2	2	42	4.1
M3	15	13	12.56	1.20	1.20			44	5.2	18	10	15.44	2	2	42	5.2
Park-Thompson (75)																
T3	16	12	13.69	1.20	1.20			60	5.4	18	9	15.56	2.79	2.79	40	5.4
Smith (17)																
S4	15	13	12.56	1.20	1.20			40	3.0	18	10	15.44	2	2	43	3.0
S5	↓	↓	↓	↓	↓			40	2.9	↓	↓	↓	↓	↓	44	2.9
S6	↓	↓	↓	↓	↓			43	2.6	↓	↓	↓	1.99	2.08	40(top) 43(bot)	2.6

Joint				Lateral Beam										
Joint Reinf.	ρ_s	f_y (ksi)	f'_c (ksi)	w_L (in.)	h_L (in.)	Column Load (kips)	Type of Joint	Failure Mode 1st Cycle	Q_t (kips)	Q_c (kips)	Q_t/Q_c	Q^*_t (kips)	Q^*_c (kips)	Q^*_t/Q^*_c
(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)	(28)	(29)	(30)	(31)
None	--	--	5.70	12	20[1]	640	E	F	164.0	205.0	0.80	145.4	164.8	0.88
↓	--	--	6.07	↓	20[1]	↓	I	↓	223.0	213.0	1.05	202.8	166.2	1.21
↓	--	--	3.81	↓	20[2]	↓	↓	↓	220.6	206.6	1.07	192.8	153.7	1.25
5-#4 @ 4.1"	3.0	45	5.69	↓	20[1]	↓	↓	↓	274.9	222.7	1.23	243.3	200.5	1.21
2-#4 @ 6"	1.1	59	3.80	--	None	357	I	S	245.0	309.0	0.79	191.0	248.4	0.77
↓	↓	↓	6.06	--	↓	360	↓	↓	359.0	338.7	1.06	224.0	268.2	0.84
↓	↓	↓	3.86	--	↓	356	↓	↓	276.0	298.0	0.93	213.0	243.1	0.88
↓	↓	↓	5.23	--	↓	363	↓	↓	327.0	296.0	1.10	255.0	214.3	1.19
↓	↓	↓	5.20	--	↓	48	↓	↓	344.0	242.1	1.42	241.0	187.9	1.28
↓	↓	↓	5.33	--	↓	603	↓	F	370.0	279.5	1.32	225.0	219.1	1.03
↓	↓	↓	5.40	--	↓	597	↓	S	330.0	278.0	1.19	240.0	199.0	1.20
↓	↓	↓	4.80	15	15[2]	355	↓	F	381.0	261.5	1.46	365.0	213.8	1.70
↓	↓	↓	4.50	8	15[2]	367	↓	S	359.0	327.3	1.10	279.0	225.2	1.23
↓	↓	↓	4.29	↓	↓	359	↓	↓	332.0	325.1	1.02	244.0	219.8	1.11
↓	↓	↓	3.72	↓	↓	365	↓	↓	289.0	243.8	1.18	250.0	158.0	1.58
6-#5 @ 2"	0.5	61	5.10	--	None	363	↓	F	438.0	285.7	1.53	313.0	197.0	1.59
6-#4 @ 2"	3.3	59	5.99	--	↓	353	↓	S	350.0	377.0	0.93	253.0	271.0	0.93
6-#4 @ 2"	3.3	59	4.81	--	↓	363	↓	S	346.0	289.2	1.20	238.0	178.2	1.33
3-#4 @ 5"	1.5	46	4.11	--	None	0	E	F	71.8	83.2	0.86	--	--	--
4-#4 @ 3.6"	2.0	36	5.19	--	"	0	E	F	69.4	88.5	0.78	--	--	--
7-#5 @ 2"	5.8	44	5.40	--	None	220	I	F	230.0	227.6	1.01	129.1	193.0	0.67
5-#4 @ 2.75"	2.7	45	2.97	--	None	0	E	F	73.1	77.0	0.95	--	--	--
4-#3 @ 3.75"	2.3	45	2.92	--	↓	↓	↓	↓	74.3	75.3	0.97	--	--	--
5-#4 @ 3"	3.2	45	2.57	--	↓	↓	↓	↓	76.6	73.9	1.04	--	--	--

Specimen (Reference No.)	Column h_c (in.)	Column b_c (in.)	Column d_c (in.)	Column A_{s1} (in.^{2})	Column A_{s2} (in.^{2})	Column A_{s3} (in.^{2})	Column A_{s4} (in.^{2})	Column f_y (ksi)	Column f'_c (ksi)	Beam h_b (in.)	Beam b_b (in.)	Beam d_b (in.)	Beam A_{st} (in.^{2})	Beam A_{sb} (in.^{2})	Beam f_y (ksi)	Beam f'_c (ksi)
(0)	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
Higashi (32)																
SR24Aa-1	7.87	11.87	6.69	0.39	0.39			43	6.1	11.81	7.87	11.02	0.49	0.49	43	6.1
SR24Aa-2	↓	↓	↓	↓	↓			↓	6.1	↓	↓	↓	↓	↓	↓	6.1
SR24Aa-3	↓	↓	↓	↓	↓			↓	4.4	↓	↓	↓	↓	↓	↓	4.4
SR24Ab-1	↓	↓	↓	↓	↓			↓	6.1	↓	↓	↓	↓	↓	↓	6.1
SR24Ba-1	↓	↓	↓	↓	↓			↓	4.4	↓	↓	↓	↓	↓	↓	4.4
SR24Ca-1*	↓	↓	↓	↓	↓			↓	6.1	↓	↓	↓	↓	↓	↓	6.1
SR24Ca-2*	↓	↓	↓	↓	↓			↓	6.1	↓	↓	↓	↓	↓	↓	6.1
SR24Cb-1*	↓	↓	↓	↓	↓			↓	6.1	↓	↓	↓	↓	↓	↓	6.1
SD35Aa-1	↓	↓	↓	0.37	0.37			57	4.6	↓	↓	↓	0.37	0.37	57	4.6
SD35Aa-2	↓	↓	↓	↓	↓			↓	4.6	↓	↓	↓	↓	↓	↓	4.6
SD35Aa-3	↓	↓	↓	↓	↓			↓	4.4	↓	↓	↓	↓	↓	↓	4.4
SD35Ab-1	↓	↓	↓	↓	↓			↓	4.6	↓	↓	↓	↓	↓	↓	4.6
SD35Ba-1	↓	↓	↓	↓	↓			↓	4.4	↓	↓	↓	↓	↓	↓	4.4
SD35Ca-1*	↓	↓	↓	↓	↓			↓	4.6	↓	↓	↓	↓	↓	↓	4.6
SD35Ca-2*	↓	↓	↓	↓	↓			↓	4.6	↓	↓	↓	↓	↓	↓	4.6
SD35Cb-1*	↓	↓	↓	↓	↓			↓	4.6	↓	↓	↓	↓	↓	↓	4.6
*Joint with slab.																
Sekine-Ogura (33)																
No. 1	9.84	9.84	8.46	0.62	0.62			51	3.8	11.02	7.08	9.45	0.89	0.89	50	3.8
No. 2	↓	↓	↓	↓	↓			↓	3.8	↓	↓	↓	↓	↓	↓	3.8
No. 3	↓	↓	↓	↓	↓			↓	4.5	↓	↓	↓	↓	↓	↓	4.5
No. 4	↓	↓	↓	↓	↓			↓	4.6	↓	↓	↓	↓	↓	↓	4.6
No. 5	↓	↓	↓	↓	↓			↓	4.7	↓	↓	↓	↓	↓	↓	4.7
Bessho (34)																
J1	15.75	15.75	13.78	2.40	1.20	1.20	2.40	55	4.6	14.76	10.82	11.81	2.4(D22) 1.78(D19)	2.4(D22) 1.78(D19)	55 54	4.6
J2	↓	↓	↓	↓	↓	↓	↓	↓	4.6	↓	↓	↓	↓	↓	↓	4.6
J3	↓	↓	↓	↓	↓	↓	↓	↓	4.9	↓	↓	↓	↓	↓	↓	4.9

Joint Reinf. (17)	ρ_s (18)	f_y (ksi) (19)	f'_c (ksi) (20)	Lateral Beam w_L (in.) (21)	Lateral Beam h_L (in.) (22)	Column Load (kips) (23)	Type of Joint (24)	Failure Mode 1st Cycle (25)	Q_t (kips) (26)	Q_c (kips) (27)	Q_t/Q_c (28)	Q_t^* (kips) (29)	Q_c^* (kips) (30)	Q_t^*/Q_c^* (31)
6-φ4.1 @ 1.57"	0.1	40	6.13	--	None	53.14	I*	F	35.1	48.4	0.71	--	--	--
↓	↓	↓	6.13	--	↓	26.57	↓	↓	35.9	36.4	0.99	--	--	--
↓	↓	↓	4.42	--	↓	26.57	↓	↓	35.9	34.9	1.03	--	--	--
↓	↓	↓	6.13	--	↓	53.14	I	↓	36.7	35.7	1.03	--	--	--
↓	↓	↓	4.42	3.94	11.8[2]	26.57	I*	↓	35.1	33.7	1.04	--	--	--
↓	↓	↓	6.13	↓	↓	53.14	↓	↓	38.9	47.0	0.83	--	--	--
↓	↓	↓	6.13	↓	↓	53.14	↓	↓	39.3	44.6	0.88	--	--	--
↓	↓	↓	6.13	↓	↓	26.57	I	↓	38.9	35.4	1.10	--	--	--
↓	↓	↓	4.61	--	None	53.14	I*	↓	38.6	45.2	0.85	--	--	--
↓	↓	↓	4.61	--	↓	26.57	↓	↓	38.2	34.0	1.12	--	--	--
↓	↓	↓	4.42	--	↓	26.57	↓	↓	38.9	50.8	0.77	--	--	--
↓	↓	↓	4.61	--	↓	53.14	I	↓	38.2	46.0	0.83	--	--	--
↓	↓	↓	4.42	3.94	11.8[2]	53.14	I*	↓	39.5	43.6	0.91	--	--	--
↓	↓	↓	4.61	↓	↓	26.57	↓	↓	40.8	31.5	1.29	--	--	--
↓	↓	↓	4.61	↓	↓	26.57	↓	↓	39.4	32.0	1.23	--	--	--
↓	↓	↓	4.61	↓	↓	53.14	I	↓	39.2	40.9	0.96	--	--	--
3-φ6 @ 3"	0.07	41	3.82	7.08	11.0[2]	22	E*	F	39.2	57.0	0.69	29.3	66.7	0.63
↓	↓	↓	3.81	↓	↓	22	↓	↓	45.4	57.0	0.80	26.0	46.7	0.57
↓	↓	↓	4.50	--	None	22	E	↓	38.0	57.4	0.66	27.1	48.2	0.56
↓	↓	↓	4.63	--	↓	22	↓	↓	38.8	58.0	0.67	19.1	37.8	0.50
3-φ6 @ 3" hori & vert	↓	↓	4.68	--	↓	22	↓	↓	36.9	58.0	0.64	16.1	36.2	0.44
6-φ9 @ 2.16" circ. & square	4.0	46	4.59	--	None	88.18	I	S	351.0	325.9	1.08	351.0	260.8	1.34
↓	↓	↓	4.59	10.82	14.8[1]	88.18	↓	↓	351.0	303.4	1.16	351.0	242.7	1.44
↓	↓	↓	4.85	10.82	14.8[2]	88.18	↓	↓	375.0	269.0	1.39	375.0	216.0	1.73

Specimen (Reference No.) (0)	Column h_c (in.) (1)	b_c (in.) (2)	d_c (in.) (3)	A_{s1} (in.^{2}) (4)	A_{s2} (in.^{2}) (5)	A_{s3} (in.^{2}) (6)	A_{s4} (in.^{2}) (7)	f_y (ksi) (8)	f'_c (ksi) (9)	Beam h_b (in.) (10)	b_b (in.) (11)	d_b (in.) (12)	A_{st} (in.^{2}) (13)	A_{sh} (in.^{2}) (14)	f_y (ksi) (15)	f'_c (ksi) (16)
Yamaguchi (35)																
N22A	19.68	19.68	17.71	3.14	1.57	3.14		56	3.2 (up) 4.8 (low)	21.65	13.78	18.90	3	3	54	3.7
N22B	↓	↓	↓	↓	↓	↓		↓	2.9 (up) 4.1 (low)	↓	↓	↓	↓	↓	54	3.4
N25A	↓	↓	↓	↓	↓	↓		↓	2.8 (up) 4.0 (low)	↓	↓	↓	3.14	3.14	56	3.2
L25A	↓	↓	↓	↓	↓	↓		↓	3.8 (up) 2.4 (low)	↓	↓	↓	↓	↓	56	3.4
N29A	↓	↓	↓	↓	↓	↓		↓	2.8 (up) 4.0 (low)	↓	↓	↓	2.99	2.99	56	3.2
N35A	↓	↓	↓	↓	↓	↓		↓	2.8 (up) 4.0 (low)	↓	↓	↓	2.97	2.97	54	3.4
Tada (30)																
A	11.81	11.81	9.84	1.23	1.23			50	4.1	11.81	7.87	9.84	0.93	0.93	50	4.1
B	↓	↓	↓	↓	↓			↓	↓	↓	↓	↓	↓	↓	↓	↓
C	↓	↓	↓	↓	↓			↓	↓	↓	↓	↓	↓	↓	↓	↓
D	↓	↓	↓	↓	↓			↓	↓	↓	↓	↓	↓	↓	↓	↓
E	↓	↓	↓	2.40	2.40			55	↓	↓	↓	↓	1.80	1.80	55	↓
Kokusho (41)*																
B13-0-M	11.81	15.75	10.63	1.56	1.56			68	3.1	13.78	11.81	13	1.23	1.23	54	3.2
B13-40-M	↓	↓	↓	↓	↓			↓	↓	↓	↓	↓	↓	↓	↓	↓
B13-40-R	↓	↓	↓	↓	↓			↓	↓	↓	↓	↓	↓	↓	↓	↓
B22-0-M	↓	↓	↓	↓	↓			↓	3.0	↓	↓	12.09	1.18	1.18	54	3.0
B22-40-M	↓	↓	↓	↓	↓			↓	↓	↓	↓	↓	↓	↓	↓	↓
B22-40-R	↓	↓	↓	↓	↓			↓	↓	↓	↓	↓	↓	↓	↓	↓
*All tests--lightweight concrete																
Ogura (36)*																
SS28	9.84	9.84	8.46	0.62	0.62			53	3.7	11.02	7.09	9.05	1.18	1.18	55	3.7
SS68	↓	↓	↓	↓	↓			↓	↓	↓	↓	↓	↓	↓	↓	↓
SST28	↓	↓	↓	↓	↓			↓	↓	↓	↓	↓	↓	↓	↓	↓
SS56	↓	↓	↓	↓	↓			↓	↓	↓	↓	↓	↓	↓	↓	↓
*All tests--lightweight concrete																

Joint				Lateral Beam										
Joint Reinf. (17)	ρ_s (18)	f_y (ksi) (19)	f'_c (ksi) (20)	w_L (in.) (21)	h_L (in.) (22)	Column Load (kips) (23)	Type of Joint (24)	Failure Mode 1st Cycle (25)	Q_t (kips) (26)	Q_c (kips) (27)	Q_t/Q_c (28)	Q_t^* (kips) (29)	Q_c^* (kips) (30)	Q_t^*/Q_c^* (31)
4-ø13 @ 3.93"	1.2	47	3.72	13.78	21.65[2]	0	E*	F	158.0	147.5	1.07	93.8	110.9	0.85
↓	↓	↓	3.36	↓	↓	↓	↓	↓	159.3	143.0	1.11	133.3	102.4	1.30
↓	↓	↓	3.18	↓	↓	↓	↓	↓	162.7	145.0	1.12	111.8	109.0	1.03
↓	↓	↓	3.36	↓	↓	↓	↓	↓	158.5	149.4	1.06	95.8	119.5	0.80
↓	↓	↓	3.22	↓	↓	↓	↓	↓	136.0	145.9	0.93	138.7	109.7	1.26
↓	↓	↓	3.35	↓	↓	↓	↓	↓	153.1	149.3	1.03	153.1	112.3	1.36
5-ø9 @ 1.72"	2.2	46	4.12	--	None	99.2	I	F	89.9	130.0	0.69	80.6	99.3	0.81
3-ø9 @ 2.87" hori & vert	1.3	↓	↓	↓	↓	↓	↓	↓	99.4	142.0	0.70	72.5	110.7	0.65
3-ø9(h), 6-ø9(v) @ 2.87"	1.3(h) 2.7(v)	↓	↓	↓	↓	↓	↓	↓	83.3	125.0	0.67	64.0	97.5	0.66
6-ø9 @ 2.87" hort & vert	2.7	↓	↓	↓	↓	↓	↓	↓	83.3	132.0	0.63	72.5	103.0	0.70
3-ø9 @ 2.87" hort & vert	1.3	↓	↓	↓	↓	↓	↓	↓	139.5	110.0	1.27	121.3	84.0	1.44
6-ø9 @ 1.97"	1.7	50	3.16	--	None	0	I	F	107.8	90.8	1.18	--	--	--
↓	↓	↓	↓	↓	↓	105.8	↓	↓	121.8	136.8	0.89	--	--	--
↓	↓	↓		↓	↓	105.8	↓	↓	--	--	--	84.5	95.7	0.86
↓	↓	↓	3.00	↓	↓	0	↓	↓	116.2	88.1	1.31	--	--	--
↓	↓	↓	3.00	↓	↓	105.8	↓	↓	113.7	143.3	0.79	--	--	--
↓	↓	↓	3.00	↓	↓	105.8	↓	↓	--	--	--	96.3	114.6	0.84
3-ø6 @ 3.2"	0.7	80	3.67	--	None	22	E	F	57.7	46.7	1.23	--	--	--
5-ø9 @ 3.2"	1.7	39	↓	↓	↓	22	↓	↓	57.4	48.1	1.19	--	--	--
3-ø6 @ 3.2"	0.7	80	↓	↓	↓	22	↓	↓	57.7	46.7	1.23	--	--	--
5-ø6 @ 1.57"	1.4	80	↓	↓	↓	22	↓	↓	62.0	46.6	1.33	--	--	--

Specimen (Reference No.) (0)	Column h_c (in.) (1)	b_c (in.) (2)	d_c (in.) (3)	A_{s1} (in.^{2}) (4)	A_{s2} (in.^{2}) (5)	A_{s3} (in.^{2}) (6)	A_{s4} (in.^{2}) (7)	f_y (ksi) (8)	f'_c (ksi) (9)	Beam h_b (in.) (10)	b_b (in.) (11)	d_b (in.) (12)	A_{st} (in.^{2}) (13)	A_{sb} (in.^{2}) (14)	f_y (ksi) (15)	f'_c (ksi) (16)
Nakada (38)																
No. 1	31.49	21.65	29.13	2.60	2.60			51	3.6	23.62	15.75	19.68	8.31	8.31	53	3.6
No. 2	31.49	21.65	29.13	2.60	2.60			51	3.6	21.65	15.75	17.71	6.23(D41) 2.46(D32)	6.23(D41) 2.46(D32)	53 57	3.6
Hamada-Kamimura (25)																
No. 1	11.81	11.81	9.84	2.40	2.40			52	2.8	11.81	7.87	9.84	1.80	1.80	52	2.8
No. 2	↓	↓	↓	↓	↓			↓	↓	↓	↓	↓	↓	↓	↓	↓
No. 3	↓	↓	↓	↓	↓			↓	↓	↓	↓	↓	↓	↓	↓	↓
Ogura (37)																
No. 1	7.87	7.87	6.30	0.89	0.89			47	4.1	7.87	4.72	6.30	0.41	0.41	44	4.1
No. 2	↓	7.87	↓	0.89	0.89			47	↓	↓	4.72	↓	0.62	0.62	46	↓
No. 3	↓	9.84	↓	0.62	0.62			44	↓	↓	5.90	↓	0.89	0.89	47	↓
No. 4	↓	9.84	↓	0.62	0.62			46	↓	↓	5.90	↓	0.89	0.89	47	↓
Yamaguchi (42)																
S16	10.63	10.63	9.37	0.93	0.62	0.93		52	3.3	11.81	7.48	10.23	0.93	0.93	52	3.3
4P16	↓	↓	↓	↓	↓	↓		↓	3.4	↓	↓	↓	↓	↓	↓	3.4
2P16	↓	↓	↓	↓	↓	↓		↓	3.2	↓	↓	↓	↓	↓	↓	3.2
2P19	↓	↓	↓	↓	↓	↓		↓	3.2	↓	↓	↓	0.89	0.89	51	3.2
2P13	↓	↓	↓	↓	↓	↓		↓	3.3	↓	↓	↓	0.98	0.98	50	3.3
Ishibashi (39,40)																
No. 1	25.98	25.98	22.05	6.28	6.28			57	3.6	23.62	15.75	19.68	6.28	6.28	57	3.5
No. 2	↓	↓	↓	↓	↓			57	3.8	23.62	15.75	↓	9.43	9.43	57	3.8
No. 3	↓	↓	↓	↓	↓			58	4.7	23.62	23.62	↓	9.43	9.43	58	4.7
No. 4	↓	↓	↓	↓	↓			58	4.5	23.62	23.62	↓	9.43	9.43	58	4.5
Popov-Bertero (12)																
BC3	17	17	15.50	1.76	0.88	0.88	1.76	71	4.5	16	9	14.50	1.76	0.93	71	4.5
BC4	17	17	15.50	1.76	0.88	0.88	1.76	71	4.6	16	9	14.50	1.76	0.93	71	4.6
BC4E	17	17	15.50	1.76	0.88	0.88	1.76	71	5.0	16	9	14.50	1.76	0.93	71	5.0

Joint				Lateral Beam										
Joint Reinf. (17)	ρ_s (18)	f_y (ksi) (19)	f'_c (ksi) (20)	w_L (in.) (21)	h_L (in.) (22)	Column Load (kips) (23)	Type of Joint (24)	Failure Mode 1st Cycle (25)	Q_t (kips) (26)	Q_c (kips) (27)	Q_t/Q_c (28)	Q^*_t (kips) (29)	Q^*_c (kips) (30)	Q^*_t/Q^*_c (31)
4-ϕ16, 4 legs @ 4.72"	2.0	51	3.61	--	None	597	I	F	786.0	626.0	1.25	643.0	530.8	1.21
↓	↓	↓	↓	↓	↓	590	↓	↓	749.0	629.1	1.19	658.0	538.5	1.22
4-ϕ9 @ 2.36"	1.8	47	2.80	--	None	79.36	I	F	135.1	87.9	1.53	121.2	77.0	1.57
↓	↓	↓	↓	↓	↓	↓	↓	↓	121.5	90.6	1.33	97.3	81.9	1.19
									108.2	94.6	1.14	98.2	79.5	1.23
2-ϕ9 @ 3.15"	2.0	48	4.09	--	None	7.5	I	F	36.0	32.6	1.10	32.0	29.6	1.08
4-ϕ9 @ 1.57"	4.0	↓	↓	↓	↓	7.13	↓	↓	39.6	36.5	1.08	39.6	32.7	1.21
↓	3.6					0			55.3	52.0	1.06	52.8	48.3	1.09
	3.6					0			48.5	52.0	0.93	48.5	49.0	0.99
4-ϕ6 @ 2.36"	0.9	43	3.27	7.48	11.81[2]	0	E*	F	47.2	41.5	1.13	39.8	21.6	1.84
↓	↓	↓	3.40	↓	↓	↓	↓	↓	46.3	43.3	1.07	42.6	27.7	1.53
			3.17						45.8	40.8	1.12	39.1	28.4	1.37
			3.19						45.9	41.0	1.11	37.7	21.3	1.77
			3.33						48.8	42.6	1.14	20.0	27.6	0.72
5-ϕ16 @ 3.15"	1.7	52	3.51	--	None	577	I	F	649.5	584.9	1.11	649.5	467.9	1.38
10-ϕ16 @ 1.57"	3.4	52	3.75	↓	↓	↓	↓	↓	746.5	645.0	1.16	566.4	490.2	1.15
5-ϕ16 @ 3.15"	1.7	50	4.69						1009.8	673.8	1.50	768.2	566.2	1.36
5-ϕ16 @ 3.15"	1.7	50	4.52						1015.0	660.0	1.54	864.2	554.4	1.56
7-#2, 4 legs @ 2"	1.4	65	4.50	--	None	470	I	F	151.0	246.0	0.62	120.0	190.0	0.63
↓	↓	↓	4.57	↓	↓	↓	↓	↓	158.0	246.6	0.64	150.0	197.8	0.76
			4.97						166.3	253.7	0.66	127.4	204.0	0.62

Specimen (Reference No.)	Column									Beam						
	h_c (in.)	b_c (in.)	d_c (in.)	A_{s1} (in.^{2})	A_{s2} (in.^{2})	A_{s3} (in.^{2})	A_{s4} (in.^{2})	f_y (ksi)	f'_c (ksi)	h_b (in.)	b_b (in.)	d_b (in.)	A_{st} (in.^{2})	A_{sb} (in.^{2})	f_y (ksi)	f'_c (ksi)
(0)	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
Birss (23)																
B1	18	18	16	2.80	1.40	1.40	2.80	62	4.1	24	14	21.25	3.89	3.89	42	4.1
B2	18	18	16	2.80	1.40	1.40	2.80	62	4.6	24	14	21.25	3.89	3.89	42	4.6
Megget (76)																
Unit A	15	13	13.60	2.37	2.37			53	3.2	18	10	16.50	2.54	1.58	54	3.2
Unit B	15	13	13.60	2.37	2.37			53	3.2	18	10	16.50	2.54	1.58	54	3.2
Ohwada (27,28)																
JO-0	5.90	3.94	5.11	0.39	0.39			59	2.9	5.90	3.94	5.11	0.39	0.39	59	2.9
JE-0	↓	3.94	↓	0.39	0.39			59	↓	↓	3.94	↓	0.39	0.39	59	↓
JI-0	↓	3.94	↓	0.39	0.39			59	↓	↓	3.94	↓	0.39	0.39	59	↓
JO-1	↓	5.90	↓	0.59	0.39	0.59		63	↓	↓	5.90	↓	0.59	0.59	63	↓
JO-2	↓	↓	↓	↓	↓	↓		↓	↓	↓	↓	↓	↓	↓	↓	↓
JE-1	↓	↓	↓	↓	↓	↓		↓	↓	↓	↓	↓	↓	↓	↓	↓
JE-2	↓	↓	↓	↓	↓	↓		↓	↓	↓	↓	↓	↓	↓	↓	↓
J1-1	↓	↓	↓	↓	↓	↓		↓	↓	↓	↓	↓	↓	↓	↓	↓
J1-2	↓	↓	↓	↓	↓	↓		↓	↓	↓	↓	↓	↓	↓	↓	↓
Beckingsale (20)																
B11	18	18	16.30	2.40	1.20	1.20	2.40	61	5.2	24	14	21.7 (top) 22.4 (bot)	3.52	1.76	43	5.2
B12	↓	↓	↓	↓	↓	↓	↓	↓	5.0	↓	↓	21.65	2.64	2.64	↓	5.0
B13A	↓	↓	↓	↓	↓	↓	↓	58	4.6	↓	↓	21.65	↓	↓	↓	4.6
Research Group of China (47)†																
JA-3	13.78	11.81	12.40	0.62	0.62	--		52	2.3	15.75	11.8 (top) 7.09 (bot)	14.37	1.18	0.79	51 (top) 52 (bot)	2.3
JA-3A(I)	↓	↓	↓	↓	↓	--		↓	2.3	↓	↓	↓	0.97	↓	63 (top) 52 (bot)	2.3
JA-3A(II)	↓	↓	↓	↓	↓	--		↓	2.7	↓	↓	↓	↓	↓	↓	2.7
KJ-1	17.71	17.71	16.34	1.37	0.35	1.37		63	2.4	25.59	11.81	24.21	1.77	1.56	51	2.4
KJ-2	↓	↓	↓	↓	↓	↓		↓	↓	↓	↓	↓	↓	↓	↓	1.7
KJ-3	↓	↓	↓	1.56	↓	1.56		↓	2.1	↓	↓	↓	↓	↓	↓	3.2
KJ-4	↓	↓	↓	1.18	↓	1.18		52	3.3	↓	↓	↓	↓	↓	↓	2.5
KJM-1(I)	9.84	9.84	8.46	0.37	0.37	--		34	3.1	13.78	5.90	12.40	0.93	0.35	52 (top) 59 (bot)	2.2
KJM-2(II)	↓	↓	↓	↓	↓	--		↓	↓	↓	↓	↓	↓	↓	↓	↓
KJM-2(I)	↓	↓	↓	0.71	0.71	--		↓	↓	↓	↓	↓	↓	↓	↓	↓

†See key

Joint				Lateral Beam										
Joint Reinf. (17)	ρ_s (18)	f_y (ksi) (19)	f'_c (ksi) (20)	w_L (in.) (21)	h_L (in.) (22)	Column Load (kips) (23)	Type of Joint (24)	Failure Mode 1st Cycle (25)	Q_t (kips) (26)	Q_c (kips) (27)	Q_t/Q_c (28)	Q^*_t (kips) (29)	Q^*_c (kips) (30)	Q^*_t/Q^*_c (31)
4-φ6.5, 4 legs @ 4.63"	0.6	58	4.06	--	None	70	I	F	268.0	173.5	1.54	193.3	150.0	1.29
4-φ6.5, 4 legs @ 4.63"	0.6	58	4.57	--	None	649	I	F	270.0	286.7	0.94	192.8	252.3	0.76
φ13 @ 1.97"	2.2	46	3.20	--	None	44.3	E	F	118.5	118.2	1.00	--	--	--
φ13 @ 1.97"	2.2	46	3.20	10	15[2]	44.3	E	F	118.0	124.0	0.95	--	--	--
None	--	--	2.93	--	None	0	E	S	20.0	21.0	0.95	11.9	16.0	0.75
↓	↓	↓	↓	5.90	5.90[1]	↓	↓	↓	19.8	21.0	0.94	11.9	16.0	0.75
↓	↓	↓	↓	5.90	5.90[2]	↓	↓	↓	24.0	24.2	1.00	24.0	18.3	1.30
3-φ4 @ 1.57"	1.2	66	2.91	--	None	↓	I	↓	34.0	31.5	1.08	22.3	23.0	0.97
↓	↓	↓	↓	--	None	↓	↓	↓	37.3	31.5	1.18	31.1	23.3	1.33
↓	↓	↓	↓	5.90	5.90[1]	↓	↓	↓	37.3	31.5	1.18	31.1	23.3	1.33
↓	↓	↓	↓	↓	5.90[1]	↓	↓	↓	37.3	31.5	1.18	37.3	23.2	1.60
↓	↓	↓	↓	↓	5.90[2]	↓	↓	↓	44.8	36.3	1.23	44.8	26.7	1.67
↓	↓	↓	↓	↓	5.90[2]	↓	↓	↓	44.8	36.3	1.23	44.8	26.7	1.67
4-#4 @ 2.2" (each direction)	4.8	49	5.20	--	None	70	I	F	210.4	206.5	1.02	201.0	176.0	1.14
4-#4 @ 2.2" (each direction)	↓	↓	5.02	↓	↓	↓	↓	↓	222.4	205.3	1.08	197.7	175.0	1.13
4-#4 @ 2.5" (each direction)	0.43	49	4.55	↓	↓	650	↓	↓	212.4	314.0	0.68	191.9	282.6	0.68
5φ □ @ 2.95"	1.2	34	2.34	--	None	66	E	F	62.1	56.0	1.10	--	--	--
↓	↓	↓	2.28	↓	↓	↓	↓	↓	63.0	54.6	1.15	--	--	--
↓	↓	↓	2.72	↓	↓	↓	↓	↓	63.0	62.4	1.00	--	--	--
7φ8 □ @ 3.9"; 7φ8 ◇ @ 3.9"	0.9	↓	2.45	↓	↓	176	↓	↓	104.3	129.0	0.81	--	--	--
↓	↓	↓	2.39	↓	↓	↓	↓	↓	104.0	127.0	0.82	--	--	--
9φ8 □ @ 2.8"; 9φ8 ◇ @ 2.8"	1.3	↓	2.11	↓	↓	↓	↓	↓	104.3	116.0	0.90	--	--	--
7φ8 □ @ 3.9"	0.5	↓	3.30	↓	↓	↓	↓	↓	104.0	153.0	0.68	--	--	--
11φ6 □ @ 1.2"; 11φ6 ◇ @ 1.2"	3.5	↓	3.06	↓	↓	66	↓	↓	52.4	47.4	1.10	--	--	--
↓	↓	↓	↓	↓	↓	↓	↓	↓	55.0	47.4	1.16	--	--	--
↓	↓	↓	↓	↓	↓	↓	↓	↓	53.0	51.4	1.03	--	--	--

Specimen (Reference No.)	Column									Beam						
	h_c (in.)	b_c (in.)	d_c (in.)	A_{s1} (in.^{2})	A_{s2} (in.^{2})	A_{s3} (in.^{2})	A_{s4} (in.^{2})	f_y (ksi)	f'_c (ksi)	h_b (in.)	b_b (in.)	d_b (in.)	A_{st} (in.^{2})	A_{sb} (in.^{2})	f_v (ksi)	f'_c (ksi)
(0)	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
U.S.S.R. (45)+																
Y-1	15.7	11.81	14.00	2.49	2.49			43	4.7	19.70	11.81	18.00	2.49	2.49	57	4.7
Y-2	↓	↓	↓	↓	↓			↓	5.1	↓	↓	↓	↓	↓	↓	5.1
Y-3	↓	↓	↓	↓	↓			↓	6.2	↓	↓	↓	3.74	3.74	↓	6.2
Y-5	↓	7.87	14.00	2.49	2.49			↓	5.1	15.70	7.87	14.00	2.49	2.49	↓	5.1
YH-1	↓	11.81	↓	3.74	3.74			↓	7.5	19.70	11.81	18.00	3.74	3.74	↓	7.5
YH-2	↓	↓	↓	3.74	3.74			↓	6.6	↓	↓	↓	↓	↓	↓	6.6
Y-I-1	↓	↓	↓	2.49	2.49			↓	5.5	↓	↓	↓	↓	↓	↓	5.5
Y-I-5	↓	↓	↓	↓	↓			↓	↓	↓	↓	↓	2.49	2.49	↓	
Y-I-6	↓	↓	↓	↓	↓			↓	↓	15.70	↓	14.00	3.74	3.74	↓	
Y-II-1	↓	↓	↓	↓	↓			↓	5.2	19.70	↓	18.00	↓	↓	↓	5.2
Y-II-4	↓	↓	↓	↓	↓			↓	5.1	↓	↓	↓	↓	↓	↓	5.1
Y-II-5	↓	↓	↓	↓	↓			↓	5.7	↓	↓	↓	2.49	2.49	↓	5.7
Y-II-6	↓	↓	↓	↓	↓			↓	3.4	↓	↓	↓	↓	↓	↓	3.4
Y-II-7	↓	↓	↓	↓	↓			↓	4.0	↓	↓	↓	↓	↓	↓	4.0
Y-II-9	↓	↓	↓	↓	↓			↓	4.6	↓	↓	↓	↓	↓	↓	4.6
Y-II-10	↓	↓	↓	↓	↓			↓	5.2	↓	↓	↓	↓	↓	↓	5.2
Y-Ⅲ-1	↓	↓	↓	↓	↓			↓	5.3	↓	↓	↓	↓	↓	↓	5.3
Y-Ⅲ-2	↓	↓	↓	↓	↓			↓	5.4	↓	↓	↓	↓	↓	↓	5.4
Y-Ⅲ-3									5.6	↓	↓	↓	↓	↓	↓	5.6

+See key.

Joint				Lateral Beam										
Joint Reinf. (17)	ρ_s (18)	f_y (ksi) (19)	f'_c (ksi) (20)	w_L (in.) (21)	h_L (in.) (22)	Column Load (kips) (23)	Type of Joint (24)	Failure Mode 1st Cycle (25)	Q_t (kips) (26)	Q_c (kips) (27)	Q_t/Q_c (28)	Q_t^* (kips) (29)	Q_c^* (kips) (30)	Q_t^*/Q_c^* (31)
None	--	--	4.69	--	None	368	I	S	379.0	417.0	0.91	--	--	--
			5.10			368			379.0	422.0	0.90			
			6.24			286			466.0	394.0	1.18			
			5.09			352			280.0	328.0	0.85			
			7.54			324; 276k††			555.0	445.0	1.25			
			6.64			308; 276k††			489.1	482.1	1.01			
Mesh	1.9		5.46			264			540.1	404.1	1.33			
	2.2		5.46			242			466.0	378.0	1.22			
	2.1		5.46			264			425.0	391.0	1.08			
	4.3		5.20			264		F	566.0	374.0	1.50			
	2.6		5.10			264		S	525.0	410.0	1.27			
	3.4		5.70			242		F	373.0	329.0	1.13			
	1.7		3.40			220		S	320.0	302.0	1.06			
	2.4		4.00			242			320.0	341.0	0.94			
	1.7		4.61			200			350.0	356.0	0.98			
	3.4		5.20			264			490.0	424.0	1.16			
None	--		5.32			264			525.0	446.0	1.18			
	--		5.43			242			487.0	448.0	1.09			
	--		5.55			242			514.0	451.0	1.14			

††Beam axial load.

APPENDIX B

DERIVATION OF SIMPLIFIED EQUATION

The equations presented in Chapter 4 are simplified herein for design application. Specifically, the aim is to eliminate the need for calculating the depth of compression zones in the beam (a_b) and the column (a_c), and the angle of inclination of the compressive strut (α).

B.1 Simplified Equations - No Beam Hinges Form

For this case, the general equation for cyclic shear strength of joints is

$$Q_c = \eta K \zeta \gamma f'_c b_c \sqrt{a_b^2 + a_c^2} \cos \alpha \qquad (5.3)$$

The geometric factors are of prime concern here. Specifically,

$$Q_m \text{ is proportional to } \lambda\, b_c \sqrt{a_b^2 + a_c^2} \cos \alpha \qquad (B.1)$$

where

$$\cos \alpha = \frac{1}{\sqrt{1 + \tan^2 \alpha}} \qquad (B.2)$$

$$\tan \alpha = \frac{h_b - 2/3\, a_b}{h_c - 2/3\, a_c} \qquad (B.3)$$

a_b = compression depth of the beam framing into a joint.

The reinforcement ratio generally lies between 0.003 and 0.05 for ρ_{min} and ρ_{max} in a beam. The corresponding value of a_b/h_b varies from 0.25 to 0.38 (see Fig. 4.3). Therefore, an average value of 0.30 for a_b/h_b can be used.

a_c = compression depth of the column framing into a joint.

a_c will be determined in accordance with two different loads.

For no axial load, the reinforcement ratio of a column is between 0.01 and 0.08. The corresponding value of a_c/h_c varies from 0.27 to 0.47 (see Fig. 4.3). Therefore, the average value for a_c/h_c can be taken as 0.35.

For axial load, a_c/h_c is obtained from Fig. 4.4. Where $M/Ph_c \geq 0.30$, take $a_c/h_c = 0.45$. Where $M/Ph_c < 0.30$, take $a_c/h_c = 0.75$. The values are intended to be conservative with regard to calculations for joint shear strength.

B.1.1 Derivation of Equations for Low Axial Load, $M/Ph_c \geq 0.30$. As compression members with $M/Ph_c \geq 0.30$ will behave in a manner similar to flexural members, they may be classified together.

Equations for No Axial Load. As mentioned before, $a_c = 0.30h_b$, $a_c = 0.35h_c$. Substituting the values for a_b and a_c selected above into Eq. (B.3),

$$\tan \alpha = \frac{h_b - 2/3\ a_b}{h_c - 2/3\ a_c} = \frac{h_b - 2/3(0.3h_b)}{h_c - 2/3(0.35h_c)} \approx \frac{h_b}{h_c} \tag{B.4}$$

Equation (B.2) becomes

$$\cos \alpha = \frac{1}{\sqrt{1 + \left(\frac{h_b}{h_c}\right)^2}} \tag{B.5}$$

Again, substituting a_b and a_c into $\sqrt{a_b^2 + a_c^2}$:

$$\sqrt{a_b^2 + a_c^2} = \sqrt{(.030h_b)^2 + (0.35h_c)^2}$$

$$= 0.35h_c \sqrt{1 + 0.75\left(\frac{h_b}{h_c}\right)^2} \tag{B.6}$$

Substituting Eqs. (B.5) and (B.6) into Eq. (B.1), we have

$$Q_m \text{ is proportional to } b_c h_c \frac{0.35\sqrt{1 + 0.75\left(\frac{h_b}{h_c}\right)^2}}{\sqrt{1 + \left(\frac{h_b}{h_c}\right)^2}}$$

or $\quad Q_m$ is proportional to $b_c h_c \lambda_{n1}$

or $\quad v_u = \frac{Q_m}{b_c h_c}$ is proportional to λ_{n1}

where

$$\lambda_{n1} = \frac{0.35\sqrt{1 + 0.75\left(\frac{h_b}{h_c}\right)^2}}{\sqrt{1 + \left(\frac{h_b}{h_c}\right)^2}}$$

= coefficient reflecting joint geometry and the M/P ratio for forces on the joint.

A plot of coefficient λ_{n1} vs. h_b/h_c is shown in Fig. B.1.

<u>Equations for axial load with $M/Ph_c \geq 0.30$.</u> Similarly for $M/Ph_c \geq 0.30$, a_c may be taken as $0.45\ h_c$

$$\tan\alpha = \frac{h_b - 2/3\ a_b}{h_c - 2/3\ a_c} = \frac{h_b - 2/3(0.3h_b)}{h_c - 2/3\ (0.45h_c)} \approx 1.1\ \frac{h_b}{h_c}$$

$$\cos\alpha = \frac{1}{\sqrt{1 + 1.21\left(\frac{h_b}{h_c}\right)^2}}$$

$$\sqrt{a_b^2 + a_c^2} = \sqrt{(0.30h_b)^2 + (0.45h_c)^2} = 0.44h_c\sqrt{1 + 0.45\left(\frac{h_b}{h_c}\right)^2}$$

$$Q_m = (1.20 - 0.10f'_c)f'_c b_c h_c \frac{0.44\sqrt{1 + 0.45\left(\frac{h_b}{h_c}\right)^2}}{\sqrt{1 + 1.21\left(\frac{h_b}{h_c}\right)^2}}$$

or $Q_m = (1.20 - 0.10f'_c)\, f'_c b_c h_c \lambda_{n2}$

or $v_u = \dfrac{Q_n}{b_c h_c} = (1.20 - 0.10f'_c)\, f'_c \lambda_{n2}$

where

$$\lambda_{n2} = \frac{0.44\sqrt{1 + 0.45\left(\dfrac{h_b}{h_c}\right)^2}}{\sqrt{1 + 1.21\left(\dfrac{h_b}{h_c}\right)^2}}$$

Coefficient λ_{n2} vs. h_b/h_c is shown in Fig. B.1.

B.1.2 Equations for Axial Load with $M/Ph_c < 0.30$. For large axial loads with $M/Ph_c < 0.30$, a_c may be taken as $0.75h_c$.

$v_u = \dfrac{Q_n}{b_c h_c}$ is proportional to λ_{n3}

where

$$\lambda_{n3} = \frac{0.75\sqrt{1 + 0.16\left(\dfrac{h_b}{h_c}\right)^2}}{\sqrt{1 + 2.56\left(\dfrac{h_b}{h_c}\right)^2}}$$

Coefficient λ_{n3} vs. h_b/h_c is shown in Fig. B.2.

B.2 Simplified Equations--Beam Hinging at Face of Column

For this case, the general equation for cyclic shear strength of joints is

$$Q_c \text{ is proportional to } b_c a_c \cos\alpha \tag{5.4}$$

$$\cos\alpha = \frac{1}{\sqrt{1 + \tan^2\alpha}}$$

$$\tan \alpha = \frac{h_b}{h_c - 2/3a_c}$$

The method of derivation of equations for this case is the same as for joints where no beam hinge occurs. Details for the derivation will not be given. Only final equations and related diagrams will be presented. However, note that $a_b = 0$ and the same a_c will be taken in corresponding situations as in Section B.1.

B.2.1 Equations for No Axial Load or Axial Load with $M/Ph_c \geq 0.30$.

Equations for no axial load

$$v_u = \frac{Q_m}{b_c h_c} \text{ is proportional to } \lambda_{b1}$$

where

$$\lambda_{b1} = \frac{0.35}{\sqrt{1 + 1.69\left(\frac{h_b}{h_c}\right)^2}}$$

Coefficient λ_{b1} vs. h_b/h_c is shown in Fig. B.3.

Equations for axial load with $M/Ph_c \geq 0.30$

$$v_u = \frac{Q_m}{b_c h_c} \text{ is proportional to } \lambda_{b2}$$

where

$$\lambda_{b2} = \frac{0.45}{\sqrt{1 + 2.04\left(\frac{h_b}{h_c}\right)^2}}$$

Coefficient λ_{b2} vs. h_b/h_c is shown in Fig. B.3.

B.2.2 Equations for Axial Load with $M/Ph_c \leq 0.30$

$$v_u = \frac{Q_m}{b_c h_c} \text{ is proportional to } \lambda_{b3}$$

where

$$\lambda_{b3} = \frac{0.75}{\sqrt{1 + 4\left(\frac{h_b}{h_c}\right)^2}}$$

Coefficient λ_{b3} vs. h_b/h_c is shown in Fig. B.4.

B.3 Simplified General Equation

After simplification, Eqs. (5.3) and (5.4) become

$$Q_c = \eta\zeta\gamma\lambda\ (1.20 - 0.10f'_c)f'_c b_c h_c \qquad (B.7)$$

or

$$v_u = \frac{Q_c}{b_c h_c} = \eta\zeta\gamma\lambda\ (1.20 - 0.10f'_c)f'_c \qquad (B.8)$$

where

λ = coefficient reflecting stressed character of joints.

B.4 Comparison of Calculation Diagrams

Comparing the curves for λ_{n1} and λ_{n2} (Fig. B.1), and λ_{b1} and λ_{b2} (Fig. B.3), it can be seen that there is little difference between the two. Therefore, the same equation can be used for both cases. Comparing λ_{b1} and λ_{b3} in Fig. B.4 with λ_{n1} and λ_{n3} in Fig. B.2, there is an increase of about 50% and 30%, respectively. It means that on the whole, shear strength of joints where no beam hinge occurs is about 1.3 to 1.5 times greater than in joints where beam hinges occur at the joint. These relations are useful for simplifying design calculations and are utilized in Chapter 6.

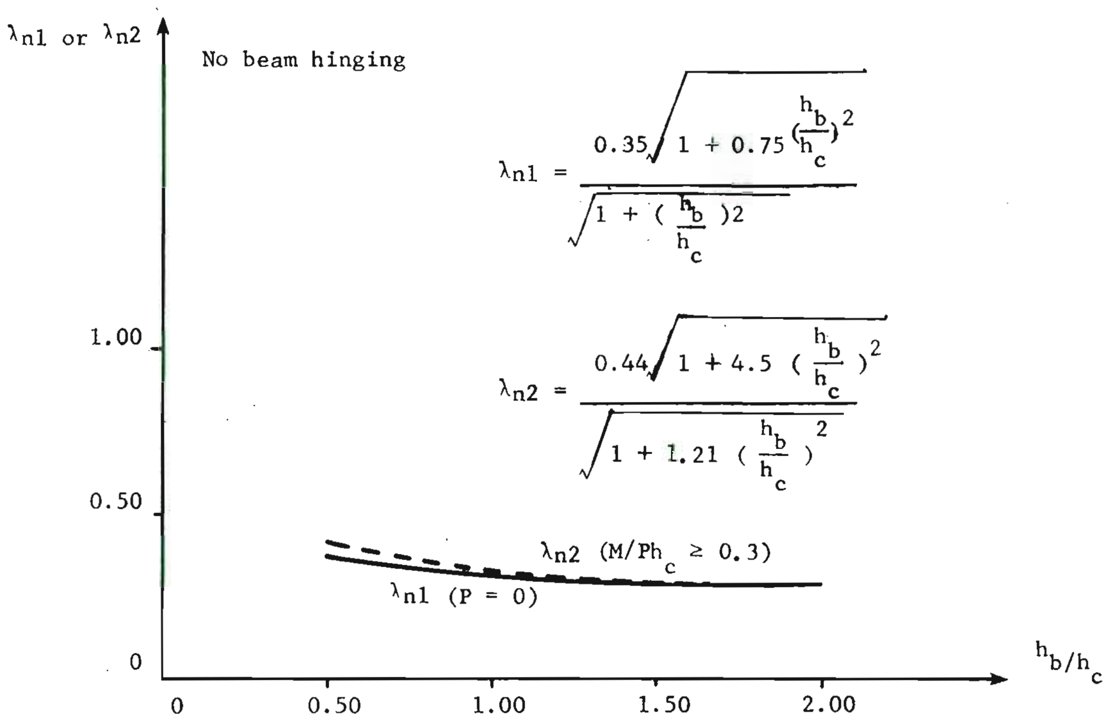


Fig. B.1 Variation of λ_{n1} and λ_{n2} with h_b/h_c

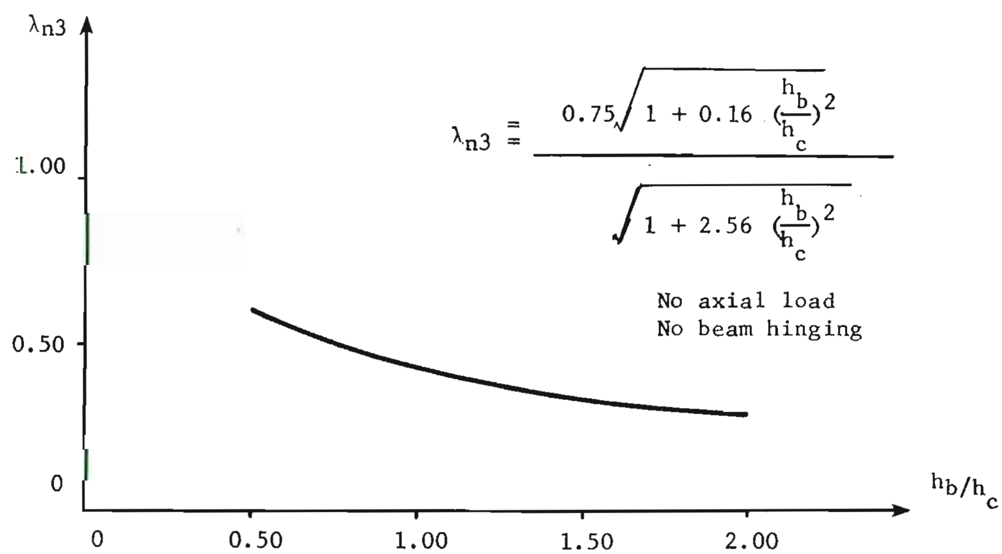


Fig. B.2 Variation of λ_{n3} with h_b/h_c

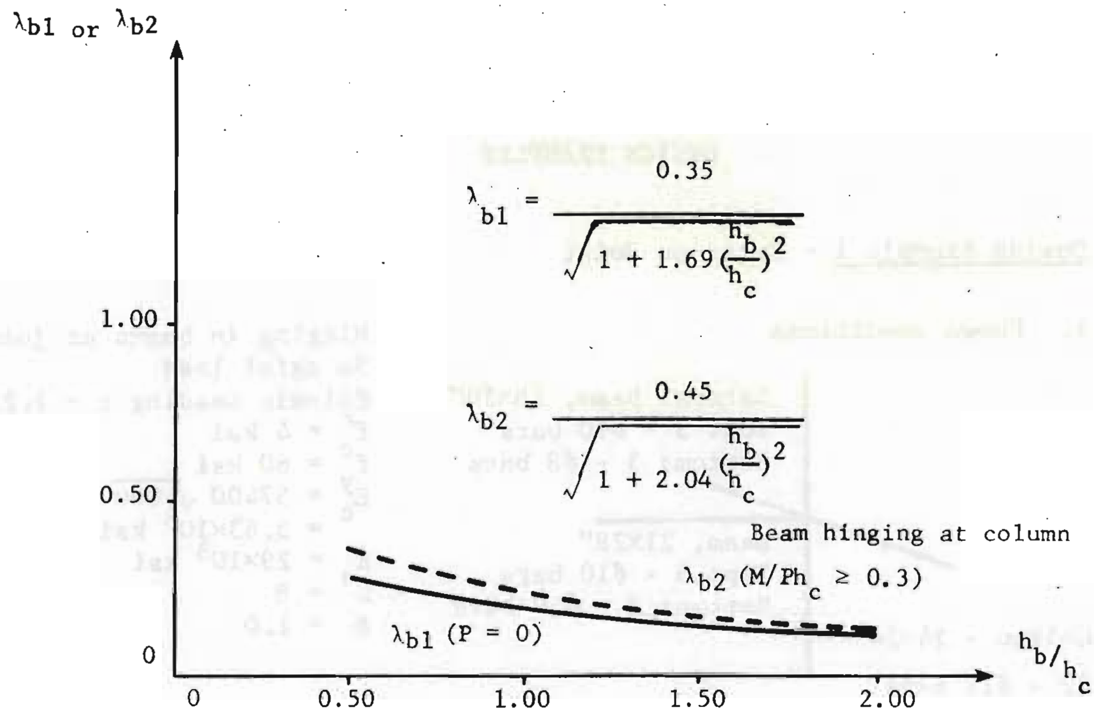


Fig. B.3 Variation of λ_{b1} and λ_{b2} with h_b/h_c

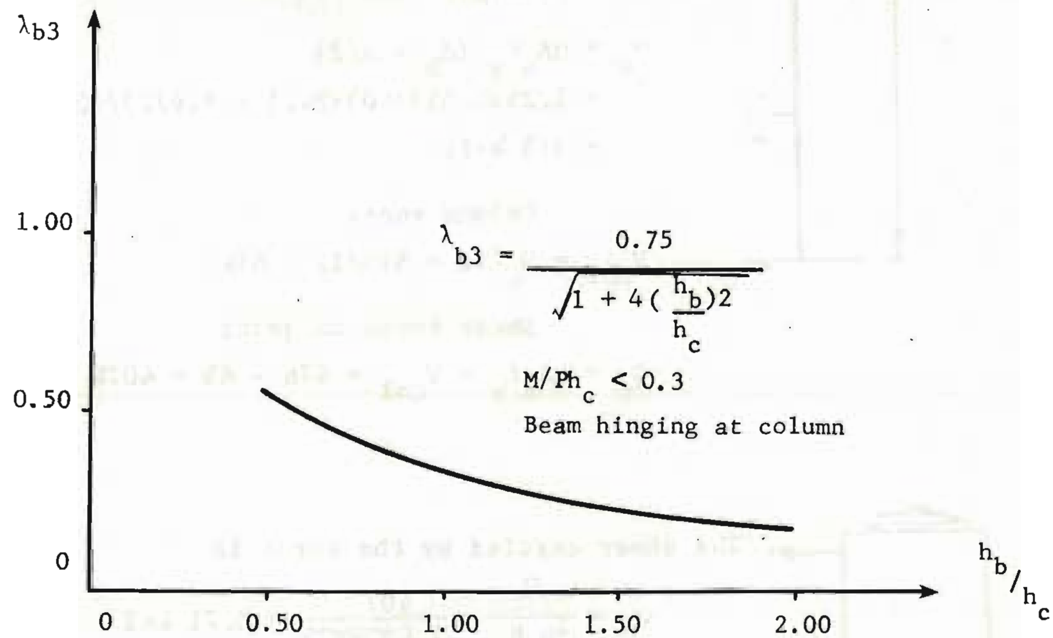


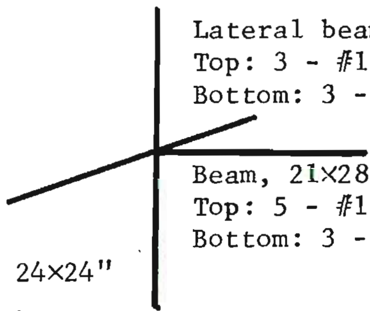
Fig. B.4 Variation of λ_{b3} with h_b/h_c

A P P E N D I X   C

DESIGN EXAMPLES

Design Example 1 - Exterior Joint

1. Known conditions



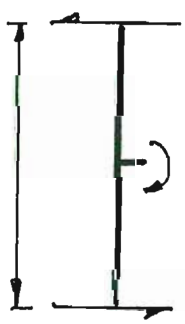
Lateral beam, 18×30"
Top: 3 - #10 bars
Bottom: 3 - #8 bars

Beam, 21×28"
Top: 5 - #10 bars
Bottom: 3 - #10 bars

Column - 24×24"
12 - #11 bars vertical

Hinging in beams at joint
No axial load
Seismic Loading α = 1.25
f'_c = 4 ksi
f_y = 60 ksi
E_c = $57400\sqrt{4000}$
$= 3.63\times10^3$ ksi
E_s = 29×10^3 ksi
n = 8
ϕ = 1.0

2. Compute shear force on joint



Singly reinforced beam moment

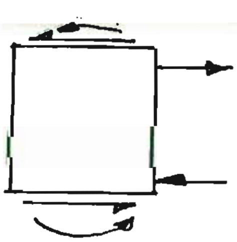
$$M_u = \alpha A_s f_y (d_b - a/2)$$
$$= 1.25(6.35)(60)(24.1 - 6.6/2)/12$$
$$= 825 \text{ k-ft}$$

Column shear

$$V_{col} = M_u/12 = 825/12 = 69\text{k}$$

Shear force on joint

$$Q_c = \alpha A_s f_y - V_{col} = 476 - 69 = 407\text{k}$$



The shear carried by the strut is

$$v_u = \frac{Q_c}{\phi b_c h_c} = \frac{407}{1.0\times24\times24} = 0.71 \text{ ksi}$$

3. Coefficient γ

For W_L = 18 in., h_c = 24 in., using Eq. (4.12)

$$\gamma = 0.85 + 0.30\ W_L/h_c = 6.85 + 0.30 \times 18/25 = 1.08$$

4. Coefficient λ

h_b/h_c = 28/24 = 1.17. Checking Fig. B.3, λ = 0.18

From Table 6.2, β = 0.35 and λ (Eq. 6.5) = 0.14.

Using Eq. (6.4), we have

$$v_u = \gamma\lambda\ (1.20 - 0.10f'_c)f'_c$$

$$= 1.08 \times 0.14 \times (1.20 - 0.10 \times 4) \times 4$$

$$= 0.49 \text{ ksi} < \frac{Q_c}{\phi b_c h_c} = 0.71 \text{ ksi}$$ N.G.

5. Modification and check

Increase column size to 27 in., then we have

$$\gamma = 0.85 + 0.30\ \frac{18}{27} = 1.05$$

6. Minimum transverse reinforcement

For #4 perimeter hoops (n = 2, A_t = 0.20 in.^{2}), and b* = h* = 20 in. using Eq. (4.11)

$$\rho_s = 2\frac{nA_t}{b^*s_t} = 2 \times \frac{2 \times 0.20}{23 \text{ X } s_t} = 0.01 \text{ (Min. } \rho_2)$$

S_t = 3.5 in. $\leq S_{max}$ from Table 6.2 O.K.

$$\frac{h_b}{h_c} = \frac{28}{27} = 1.04 \quad \therefore \quad \lambda = 0.16 \text{ [}\beta \text{ from Table 6.2 and Eq. (6.5)]}$$

From Eq. (6.4), we write

$$v_u = \gamma\lambda\ (1.20 - 0.10f'_c)f'_c$$

$$= 1.05 \times 0.16 \quad (1.20 - 0.10 \times 4) \times 4$$

$$= 0.54 \text{ ksi} \approx \frac{Q_c}{\phi b_c h_c} = \frac{407}{1.0 \times 27 \times 27} = 0.56 \text{ ksi} \qquad \text{O.K.}$$

Design Example 2 - Exterior Joint

1. Known conditions

All the known conditions are the same as Example 1, except that axial load $P/A_g = 1000$ psi, i.e., P = 576k ($P = 0.18P_o$)

2. Coefficient γ

As in Example 1 (column 24 × 24 in.)

3. Coefficient λ

$$M = V_{col} \frac{(H - h_b)}{2} = 69 \times \frac{(12 \times 12 - 28)}{2} = 4002 \text{ k-in.}$$

$$M/Ph_c = \frac{4002}{576 \times 24} = 0.27 < 0.30, \qquad h_b/h_c = \frac{28}{24} = 1.17$$

From Table 6.2 $\beta = 0.55$ and from Eq. (6.5) $\lambda = 0.22$

4. Shear strength under seismic loading

From Eq. (6.4), we have

$$v_u = \gamma\lambda\ (1.20 - 0.10f'_c)f'_c$$

$$= 1.08 \times 0.22\ (1.20 - 0.10 \times 4) \times 4$$

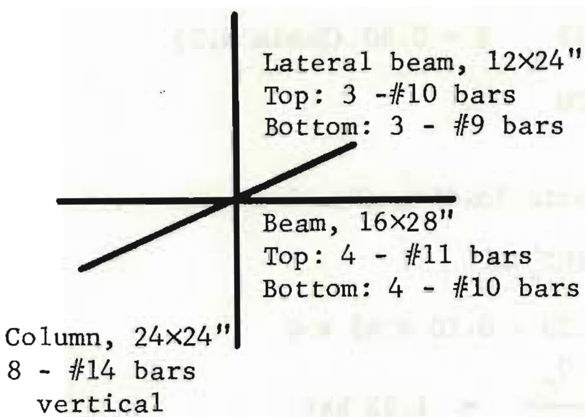
$$= 0.76 \text{ ksi} > \frac{Q_c}{\phi b_c h_c} = 0.71 \text{ ksi} \qquad \text{O.K.}$$

No increase in size required.

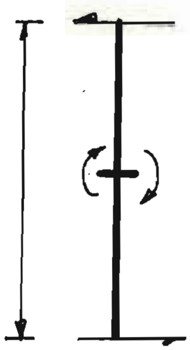
Design Example 3 - Interior Joint

1. Known conditions

No beam hinge
No axial load
Seismic loading $\alpha = 1.25$
$f'_c = 4$ ksi
$f_y = 60$ ksi
$E_c = 3.63\times10^3$ ksi
$E_s = 29\times10^3$ ksi
$n = 8$
$\phi = 1.0$



2. Compute shear forces on joint



Singly reinforced beam moments:

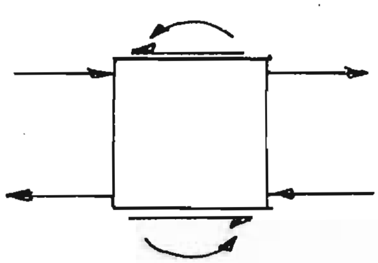
$$M_{ut} = 1.25 \times 6.24 \times 60(24 - 8.6/2)/12 = 770 \text{ k-ft}$$

$$M_{ub} = 1.25 \times 5.08 \times 60 \times (25.5 - 7/2)/12 = 700 \text{ k-ft}$$

$$V_{col} = \frac{M_{ut} + M_{ub}}{12} = \frac{770 + 700}{12} = 120\text{k}$$

Shear forces on the joint:

$$Q_c = 470 + 380 - 120 = 730\text{k}$$



$$v_u = \frac{Q_c}{\phi b_c h_c} = \frac{730}{1 \times 20 \times 24}$$

$$= 1.52 \text{ ksi}$$

Note that $b_b = 16$ in. $< 0.75 b_c = 0.75 \times 24 = 18$ in.
Use $(16 + 24)/2 = 20$ in.

3. Coefficient γ

For W_L = 12 in., h_c = 24 in.

$$\gamma = 0.85 + 0.30 \frac{W_L}{h_c} = 0.85 + 0.30 \frac{12}{24} = 1.0$$

4. Coefficient λ

h_b/h_c = 28/24 = 1.17 $\quad \beta$ = 0.50 (Table 6.2)

From Eq. 6.5, λ = 0.20

5. Shear strength under seismic loading (Eq. 6.4)

$$v_u = \gamma\lambda\ (1.20 - 0.10f'_c)f'_c$$

$$= 1.0 \times 0.20\ (1.20 - 0.10 \times 4) \times 4$$

$$= 0.65 \text{ ksi} < \frac{Q_c}{\phi b_c h_c} = 1.52 \text{ ksi} \qquad \text{N.G.}$$

6. Modification

Increase concrete strength to 5 ksi and column size to 32 in.

h_b/h_c = 28/32 = 0.88, β remains 0.50

and λ = 0.24, γ = 1.0

From Eq. (6.4) with

$$v_u = 1.0 \times 0.25 \times (1.20 - 0.10 \times 5) \times 5 = .88$$

$$v_u < \frac{Q_c}{\phi b_c h_c} = \frac{730}{1.0 \times 24 \times 28} = 0.95 \text{ ksi (difference is < 8\%)} \qquad \text{O.K.}$$

Use $(b_c + b_b)/2$ = (32 + 16)/2 = 24

7. Minimum transverse reinforcement

For #5 hoops and ρ_s (min) = 0.01

$$\rho_s = 2 \frac{n\ A_t}{b^* t}\ 2 \times \frac{4 \times 0.31}{28 \times s_t} = 0.01$$

s_t = 4.42 in., use s_{max} from Table 6.2 = 4 in.

Design Example 4 - Interior Joint

1. Known conditions

All the known conditions are the same as Example 3 except axial load $P/A_g \approx 2100$ psi, i.e., $P = 1200k$ ($P = 0.40P$) and hinging in beam at joint is permitted. Start with 30 × 30 in. Column.

2. Coefficient γ

$$\gamma = 1.0 \text{ since } W_L < 0.5\ h_c$$

3. Coefficient λ

$$M = V_{col} \frac{(H - h_b)}{2} = 120 \times \frac{(12 \times 12 - 28)}{2} = 6960 \text{ k-in.}$$

$$M/Ph_c = \frac{6960}{1200 \times 30} = 0.19 < 0.30 \qquad h_b/h_c = 28/30 = 0.93$$

From Table 6.2, $\beta = 0.55$, and from Eq. (6.5), $\lambda = 0.26$

4. Shear strength under seismic loading

From Eq. (6.4)

$$v_u = \gamma\lambda\ (1.20 - 0.10f'_c)f'_c$$

$$= 1.0 \times 0.25\ (1.20 - 0.10 \times 4) \times 4$$

$$= 0.83 \text{ ksi} < \frac{Q_c}{\phi b_c h_c} = \frac{730}{1.0 \times \frac{(30 + 16)}{2} \times 30} = 1.06 \text{ ksi} \qquad \text{N.G.}$$

5. Modification and check

Increase column size to 32 in., and f'_c to 5 ksi

$h\ /h\ = 28/32 = 0.88$, β from Table 6.2 = 0.55

$\lambda = 0.27$ from Eq. (6.5)

$$\text{and } v_u = 1.0 \times 0.27\ (1.2 - 0.10 \times 5) \times 5 \doteq 0.95 < \frac{730}{1.0\ \frac{(32 + 16)}{2} \times 32} = 0.95$$

O.K.

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