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State-of-the Art Report
SEISMIC BEHAVIOUR OF BEAM-COLUMN JOINTS IN REINFORCED CONCRETE SPACE FRAMES

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SUMMARY

The state-of-the-art in general, together with diverse approaches to the definition of design and performance criteria of beam-column joints in ductile reinforced concrete frames, are reviewed. This is followed by more detailed discussion of identified critical aspects of joint behaviour under likely seismic actions. Against this background the highlights of findings derived from recent experimental studies, presented at this conference, are reported and compared. As a conclusion, areas in which further research appears to be desirable are suggested.

INTRODUCTION

Some twenty years ago very little, if any, attention was paid to the design of joints in reinforced concrete frames. This is surprising because at an early stage of the design of steel framed structures, engineers focussed their attention to beam-column joints. Usually causes of failure in reinforced concrete frames, which were seriously damaged or which collapsed during recent earthquakes, could be attributed to unsuitable energy dissipating mechanisms within the framing system and to poorly designed and detailed beams and columns. Because of this, joint failures during earthquakes were relatively rarely observed. Hence their importance remained largely unrecognized.

With significant progress made in the understanding and design of beams and columns, and particularly with the detailing of the reinforcement to achieve the necessary ductility in appropriate regions of these members, the importance of joints has gradually emerged. Using well designed beams and columns, a joint may well become the weakest link of the chain of resistance within a ductile reinforced concrete frame. Since the landmark publication of Hanson and Connor in 1967 [Ref.1], experimental and theoretical research commenced and progressed. The scope of this report does not allow even a cursory documentation of the world wide research contributions to this topic to be made. A state-of-the-art review, covering accessible research efforts for the first 15 years, together with proposals for simple modelling of the interplay of internal forces and those of resistance mechanisms for beam-column joints, were published in 1975 [Ref.2].

The first definitive recommendations for beam-column joints were presented in codes for the design of concrete structures in the United States [Ref.3] and New Zealand [Ref.4]. The first was the outcome of the work of ACI-ASCE Committee 352 [Ref.5], largely based on research carried out in the United States. The second was initiated by the New Zealand National Society for Earthquake Engineering, utilizing locally obtained research data. Several other countries adopted with various modifications these first code proposals or are in the process of making suitable provisions.

Design considerations and parameters affecting joint behaviour are numerous. Their interrelationship is complex, particularly in the case of space frames subjected to seismic loading, the prime subject of this report. It is thus not surprising that approaches to the design of joints, developed independently in different countries, are diverse and in some aspects conflicting. This is partly due to differences in the interpretation of experimental results when these are related to local definitions of performance or design criteria.

Some groups of researchers rely predominantly on empirical evidence and are strongly motivated by a desire to develop relatively simple and practical rules for design. Another group, while recognizing the need for simplicity, places more emphasis on the use of simplified models of behaviour, similar to those widely used for example in the prediction of the flexural or shear strength of reinforced concrete members. Missing links in such modelling, are, however, often derived from test results. Yet another school endeavours to predict the seismic response of joints solely with the aid of mathematical models, using typically finite element analysis techniques.

A recognition of emerging differences in the interpretation of tests results, definitions of performance criteria, and proposed design procedures, prompted a coordinated rather than casual cooperation between interested research groups. Of particular importance is the experimental program undertaken since 1985 and mutually agreed to by researchers in the United States, New Zealand, Japan and the People's Republic of China. Most of the relevant papers presented at this conference, are summaries of group contributions to this cooperative project.

DESIGN CRITERIA FOR BEAM-COLUMN JOINTS

Criteria suggested, but not necessarily accepted universally, for the design of joints in ductile reinforced concrete frames in seismic regions, may include the following aspects :

(1) The strength of a joint should not be less than the maximum strength of the weakest member it connects. This maximum strength is associated with the probable strength properties of the materials mobilized when the maximum ductility, which may be expected to be developed during a future earthquake, is imposed on that member. The aim of this strength hierarchy is to prevent a joint from becoming the major component of energy dissipation in a ductile frame. The principal mechanisms of load transfer within a joint, to be examined subsequently, are not considered suitable to ensure stable hysteretic response. Moreover, a higher degreee of protection against damage to joints appears to be warranted because of the difficulty of repair in a region not readily accessible.

(2) A joint, being also an integral part of a column, should not jeopardize the capacity of adjacent column sections.

(3) Joint deformations should be predictable and should not significantly reduce frame stiffness and hence affect elastic storey drifts.

(4) Joint reinforcement, necessary to ensure satisfactory performance, should not cause undue construction difficulties.

These principles may be open to critique. Some schools may not consider it necessary that quantified strength hierarchy between beams, columns and joints should be established. Others may not be overly concerned about the influence of joint deformations upon frame deformations. A test assembly, capable of sustaining its design strength after 2 to 3%, storey drift, is often considered to be acceptable, irrespective of the displacement ductility ratio involved. Some designers, however, may prefer to be able to compare inelastic displacements with those envisaged by codes and defined in terms of those displacement ductility ratios which have been used in the determination of lateral design loads. Others may wish to ensure that storey drifts during the elastic response of frames to a moderate earthquake, are comparable to those envisaged for design wind loads. Thus when the performance of test units is being evaluated, it is useful to relate findings to design criteria, such as enumerated here.

CRITICAL FEATURES OF BEHAVIOUR

The analyticial derivation of actions, introduced to joints by adjacent beams in one or two directions and by columns, is well established. These actions lead to significant shear forces within a joint in both the horizontal and vertical directions. Horizontal joint shear forces are typically 4 to 6 times larger than the shear forces simultaneously generated in adjacent columns. Significant and often the major parts of the joint shear forces are introduced by bond from steel bars to the concrete of the joint core. Therefore shear and bond strength are considered to be the two most important aspects of joint design. The discussion which follows encompasses mainly joints of space frames, typical examples of which are shown in Fig. 1.

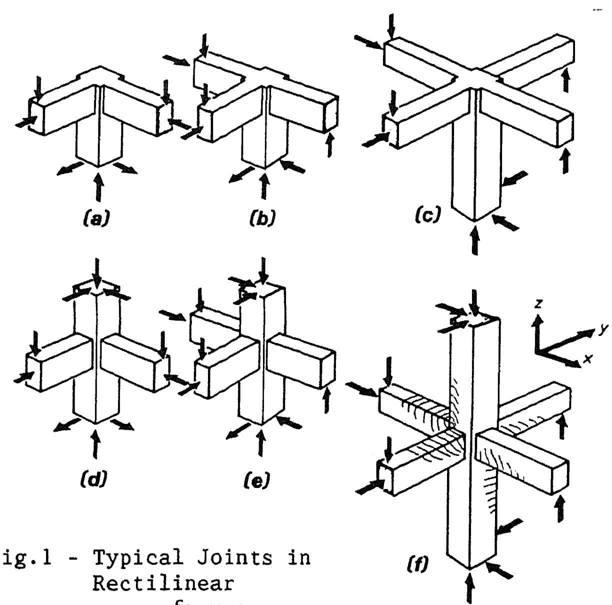

Fig.1 - Typical Joints in Rectilinear space frames.

Joint Shear Strength The principles of shear strength and associated mechanisms, used in the design of linear reinforced concrete members, are equally applicable to joints. When members around a joint contain only a small amount of flexural reinforcement or when the imposed lateral load on the frame induces small internal tension forces, the diagonal tension generated by joint shear forces may be sufficiently small so that no or very little diagonal cracking of the concrete occurs in the joint core. Such a situation is clearly not critical. The tensile strength of joint transverse reinforcement, which may have been provided, will not be mobilized. In multistorey frames, however, joint shear forces will be usually significant. As a consequence, extensive diagonal cracking of the joint core, corresponding with each direction of the lateral loading, must be expected. It has been generally recognized that the failure of a joint due to diagonal tension, leading to a corner to corner diagonal failure plane, should be prevented.

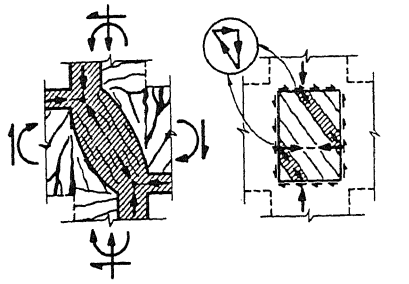

Fig.2 - Mechanisms of Shear Transfer at an Interior Beam-Column Joint of a Plane Frame.

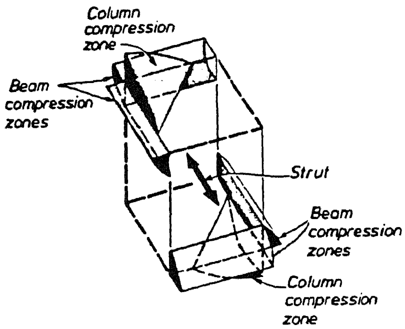

Fig.3 - End Conditions for Diagonal Strut in Space Frame Joint.

Mechanisms of shear resistance, such as shown in Fig.2, have been postulated and used as the basis of joint design [Ref.2]. Accordingly it is assumed that all compression and shear forces introduced to the joint by adjacent members by means of concrete stresses only, together with some bond forces from beam and column bars, are equilibrated by a single diagonal strut, as shown in Fig.2(a). The likely development of a principal diagonal strut in an interior joint of a space frame is shown in Fig.3. The remainder of the joint shear forces, introduced solely by steel forces by means of bond, are also assumed to develop a diagonal compression field. This mechanism, however, necessitates horizontal and usually also vertical joint reinforcement, as shown in Fig.2(b).

Some schools [Ref.3] do not rely on such modelling but prefer to use empirical evidence and performance tests to determine the desired amount of joint reinforcement. In some tests it has been found that the amount of (horizontal) joint reinforcement does not significantly influence joint shear strength. However, in such cases it was also found that a breakdown of bond, to be discussed subsequently, has occurred. Apart from limiting nominal shear stresses, the design approach adopted in the United States [Ref.3] does not emphasize shear strength and it does not use behavioural models. The primary role of horizontal joint reinforcement, consisting of stirrups, ties, hoops or spirals, is considered [Ref.3] to be confinement of the concrete in the joint core, much the same way as in the end regions of columns immediately below and above a joint. Thereby the integrity of the concrete in the joint core is assumed to be preserved, enabling it to transfer the necessary shear forces. In joints of space frames of the type shown in Fig.1(e) and (f), transverse beams are assumed to provide significant confinement to the joint core, provided that the width of the beam is at least three quarters of that of the columns. This design approach implies that joints of the type shown in Fig.1(b)(c)(e) and (f), are less critical than similar joints of plane frames. Some test results have been claimed to prove this aspect.

The design procedure in New Zealand is based on the assumption that, provided that efficient bond strength is sustained, the contribution of the concrete to joint shear strength, i.e. the mechanism shown in Fig.2(a), is gradually reduced with both the number and the magnitudes of inelastic displacement excursions during an earthquake. In certain cases it is conservatively assumed that the entire joint shear force should be resisted by the mechanism of Fig.2(b), requiring considerable amounts of transverse joint shear reinforcement. To avoid a diagonal compression failure of the concrete in the joint core, after it has been damaged by diagonal cracking along possibly four inclined planes, the maximum intensity of the diagonal compression stress should be limited. This may be achieved by limiting the nominal shear stress over an assumed effective joint area. For convenience such limits have been expressed in terms of the tensile strength of the concrete [Refs. 3 and 4] with typical values of 1.2 to 1.6 $\sqrt{f'_c}$(MPa) where f'_c is the specified compression strength of the concrete in MPa.

Failure due to shear, either by diagonal tension or compression, results in a dramatic reduction of lateral load resistance in a frame. A general consensus exists that, for this reason, shear failure in joints should be avoided. The resistance against a diagonal tension failure in joints of the type shown in Fig.1(f) may be enhanced by transverse beams because diagonal failure planes will also penetrate adjoining regions of these beams. This enhancement of shear resistance may be profound when transverse beams are not affected by earthquake actions, i.e. when they remain largely unloaded.

<u>Bond Strength</u> Most researchers recognized that for both beam and column bars, passing through interior joints, the requirements specified by codes for the development by bond of bar yield strength can not be satisfied. This necessitates compromises in the formulation of design recommendations for anchorages in joints. These also need to take into account the fact that when plastic hinges develop in both adjacent beams, steel stresses may increase to $\lambda_o f_y$, where λ_o is the overstrength factor for the particular grade of steel with values typically 1.2 to 1.5. With cycles of reversed loading of adjacent beam plastic hinges, yield penetration into the joint core and a consequent increase of the necessary bond stresses, is to be expected. Bond strength may be considered to be adequate when the overstrength of a beam bar, $\lambda_o f_y$, in both tension and compression, can be attained simultaneously with the bar remaining elastic at and near the centre of the joint core. Examples of distributions of measured steel stresses along a beam bar with diameter d_b = 28.6mm, (f_y = 275 and $\lambda_o \approx 1.23$), passing through a h_c = 686mm wide column (h_c/d_b = 24), when the test assembly was subjected to displacement ductilities of μ = 1,2,4 and 6, are shown in Fig.4. It demonstrates that, with suitable selection of bar diameter, the ideal conditions, whereby the full yield strength of beam bars in a beam plastic hinge is developed in both tension and compression, is achievable. However, conditions for this desirable bond transfer are likely to be inferior in three dimensional joints because the concrete around beam bars passing through in one direction will be subjected to significant tensile strains, introduced by similarly loaded beam bars passing through the joint at right angles. When, as a result of excessive bond stresses, caused by progressive yield penetration into the joint core or because of the use of large diameter bars, bar slip within the joint commences, both beam and joint behaviour will be affected. With the breakdown of bond, a beam bar may be subjected to tension over the entire length of the joint. In this case it will not act as compression reinforcement at the adjacent critical beam sections as intended. The internal joint forces associated with such a situation are depicted in Fig.5(a).

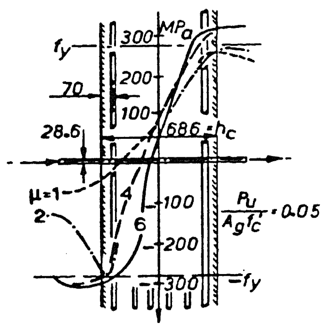

Fig.4 - Distributions of Measured Steel Stress Along a Beam Bar Passing through an Interior Joint.

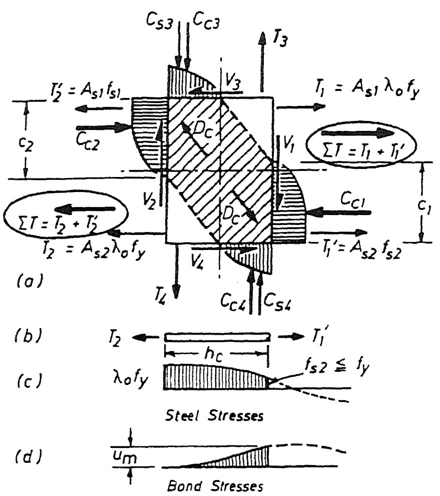

Fig.5 - Redistribution of Internal Beam Forces Resulting from the Breakdown of Bond Strength in a Joint.

For example a bar at the bottom of a beam may be subjected to tensile forces T_2 and T_1' (Fig.5(b)) because of dramatically reduced bond transfer (Fig.5(d)). As a consequence the moment at the adjacent beam section will need to be resisted by larger internal tension forces, for example $\Sigma T = T_1 + T_1'$, acting with a reduced internal lever arm. This in turn increases the magnitude of internal concrete compression force to $C_{c1} = \Sigma T$. Therefore the beneficial effect of the flexural compression reinforcement, to allow large curvature ductility to be achieved in the beam plastic hinge, is entirely lost. Important is also the fact that beam bars within the joint become significantly longer. With bar anchorage being developed primarily in the beams, rather than in the joint, the joint becomes slack. During the inelastic seismic response of a ductile frame this phenomenon manifests itself in large reduction in both stiffness and energy dissipation. Thereby some of the design criteria, set out earlier, may be jeopardized.

Of the numerous parameters affecting bond strength, the ratio of anchorage length to bar diameter, to be examined subsequently, appears to be the most significant one. Relaxation for similar ratios, relevant to column bars, have been suggested when it can be shown that a plastic hinge cannot develop in the relevant column so that column sections above and below a joint remain essentially elastic. Similar relaxation are also relevant to beam bars when their yielding at joint faces is prevented, for example by means of relocated plastic hinges [Ref.4].

<u>The Effects of Joint Behaviour on Frame Stiffness</u> As one of the performance criteria for ductile frames, most codes impose drift limits. These may be estimated, for example in terms of interstorey deflections Δ_e, calculated for an elastic frame subjected to the specified (factored) lateral static load. Alternatively values for maximum drift, including expected inelastic deformations, are specified, taking potential non-structural damage and hazards to occupants into account. Requirements for minimum frame stiffness may take the form $\mu\Delta_e/\ell_c \leq C$, where μ = displacement ductility factor with typical values of 1 to 6.5, ℓ_c = storey height. The value of C is typically in the range of 0.01 to 0.02. Thus, for example, if the drift is to be limited to 2% of the storey height when a storey displacement ductility ratio of 6 is expected, the maximum elastic displacement should be limited to 2/6 = 0.33% of the storey height. This hypothetical displacement of a perfectly elastic-plastic system provides a good indication of what the desired stiffness of a thoroughly cracked but elastic frame should be.

Contrary to traditional assumptions used in frame analyses, joint deformations due to lateral loading may be significant. Shear deformations and particularly bond slip may significantly contribute to storey drift.

This aspect should also be considered when the quality of the performance of test units is being assessed.

The Contribution of a Floor Slab If it is to be assured that a joint, such as shown in Fig.6, does not become the weakest link, than the maximum strength of the weakest member, normally the beams, must be assessed. For this reason allowance should be made for the contribution of flanges of T beams to the flexural overstrength of beams. When flanges are in compression this contribution can be shown to be negligible. However, the flexural strength of beams may increase sufficiently to warrant its consideration in both the design of joints and that of columns. For this reason in several countries considerable attention has been paid recently to this issue. At least one code requires specifically slab contribution in tension to be considered (Ref.4).

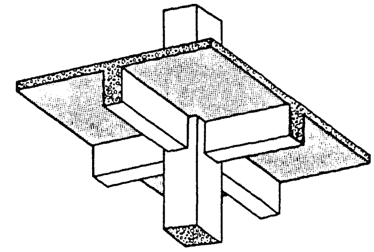

Fig.6 - A Prototype Interior Beam-Column Joint.

The primary aim of recent experimental studies was to establish the effective width of slabs, beyond the edges of columns (Fig.6), within which reinforcement, placed parallel with a beam, will contribute to its flexural strength. As expected, it was found that with increased ductility, larger quantities of slab bars, some situated a long distance away from the beam-column joint, are being mobilized. Therefore designers may need to consider this contribution at two stages, one at small ductilities ($\mu < 2$) and one at the development of maximum expected ductility ($\mu \geq 6$). To appreciate the contribution of tension flanges to load input into joints, the mechanism of internal force transfers should also be studied. Moments, developed during an earthquake in beams and columns, can equilibrate each other only at a joint. Therefore tension forces in parts of flanges, at a distance from a joint, can contribute to beam strengths only if they can be introduced to the joint core. Little attention has been paid so far to this aspect.

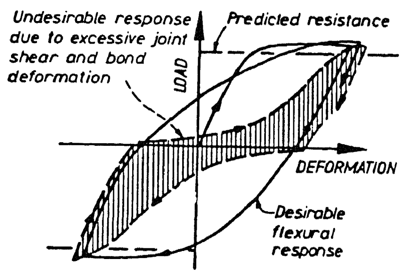

Fig.7 - A Comparison of Hysteretic Responses.

The Influence of Joints on Hysteretic Response It was stated that shear and bond are the critical actions influencing joint deformations. Even if both of these actions are well controlled, joint deformations, often neglected in frame analysis procedures, are far from being negligible. In well detailed test units of the form seen in Fig.6, typically 20% of the total elastic and inelastic deformations may originate from within joints. When bond slip is permitted to occur, large storey drifts may be required before the full lateral load resistance of a frame can be mobilized. Extreme cases of hysteretic response, one dominated by flexure and the other by shear and bond deformations, are compared in Fig.7. In order to provide significant hysteretic damping, it appears to be desirable to design and detail frames so as to enable large amounts of energy in each displacement cycles to be dissipated, and to avoid the type of response shown shaded in Fig.7. As a rule this can be easily achieved. Dynamic analyses of prototype frames showed, however, that for some earthquake records, "pinching" of hysteresis loops did not influence the displacement response significantly. As yet there is no concensus with regards to the acceptable deterioration in energy dissipation in ductile reinforced concrete structures.

Ductility Demands Mechanisms of both shear and bond resistance, illustrated in Figs. 2,4, and 5, are strongly dependent on strains imposed by an earthquake on bars passing through a joint. In particular yield penetration into the joint core may affect bond performance which in turn, as stated earlier, influences the mechanisms of joint shear resistance. Significant improvements in joint performance with reduced amounts of transverse joint reinforcement have been shown to be achievable if ductility demands, i.e. the yielding at the boundaries of a joint, for example in beam bars, is prevented. To ascertain this, plastic hinges in beams of fully ductile frames, relocated away from column faces, have been used (Ref.4). Such joints are expected to remain elastic, irrespective of the intensity of earthquake attack. Apart from the reduction of joint reinforcement, a major advantage is the dramatic improvement in the anchorage of beam bars within a joint. This enables the use of lesser number of larger diameter bars.

RECENT RESEARCH FINDINGS

Recent tests, particularly those which were part of the joint United States-New Zealand-Japan-China research project [Refs. 6,7,8,9,10,11,12], some of which have been presented during this conference, are of considerable significance because they allow meaningful comparisons to be made, even though details of design were different. The full evaluation and a detailed comparison of the wealth of data obtained, are still in progress. Only some highlights of common features, as well as notable differences, are briefly reviewed here.

Test Specimens By agreement, test specimens, constructed to approximately half [Refs. 6,9] or full [Refs, 7,8,11,12] size, had comparable strength properties. Reinforcement of the joints, which followed local code requirements, represented significant differences between specimens of the research groups. Some units modelled one-way, but most specimens simulated subassemblages of rectilinear two-way frames with beams cast together with a floor slab. Most researchers made comparisons with previously studied interior joint specimens without floor slabs. Cyclic storey displacements or equivalents, with step-by-step increase of amplitudes, A, were imposed in patterns shown in Fig.8. For one-way frames the pattern of Fig.8(a) and for space frames predominantly a combination of consecutive displacement patterns shown in Fig.8(a)(b)(e)(f) were used. Displacement increments were either in terms of displacement ductility μ, or storey drift (%), as seen for example in Fig.9. Small or no axial compression load was imposed on columns.

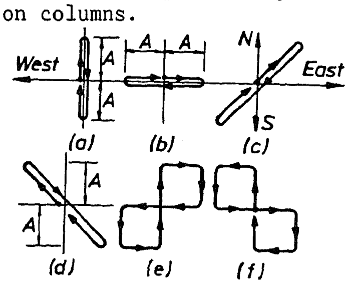

Fig.8 - Plan View of Typical Storey Displacement Patterns Used in Recent Tests.

Observed Failure Modes All reported units developed computed strengths till very large displacements, i.e. drifts of up to 4%, were imposed. At this stage either joints or beam plastic hinges failed, or tests were simply terminated.

Hysteretic Response The influence of joint distortions was best demonstrated by the shapes of the hysteretic loops. A greater degree of degradation, in terms of the ability of an assembly to dissipate energy, was observed in all reported test, when performances were compared with those of plane frame specimens. While units containing relatively small amounts of transverse joint reinforcement repeatedly developed high strength, degradation of stiffness at low loads (pinching) due to bond deformations within the joint was more significant in these units. A reduction of lateral load resistance, developed in a particular direction of imposed peak displacement, was observed in all specimens, when displacements, in accordance with the pattern of Fig.8(e) or (f), commenced at right angles to the original loading. Thus the full strength of the units in each principal direction could not be developed simultaneously. A comparison of the performance of two very similar units with slabs, one being a plane frame (Fig.9(a)) and the other a space frame ((Fig.9(b)) specimen, shows typical differences resulting from one-way and two-way actions. Unit 1D-I was subjected to the displacement pattern shown in Fig.8(a). For Unit 2D-I only the response in the North-South direction is shown while the imposed displacements were predominantly in accordance with the pattern of Fig.8(e) and (f). Relatively good energy dissipation

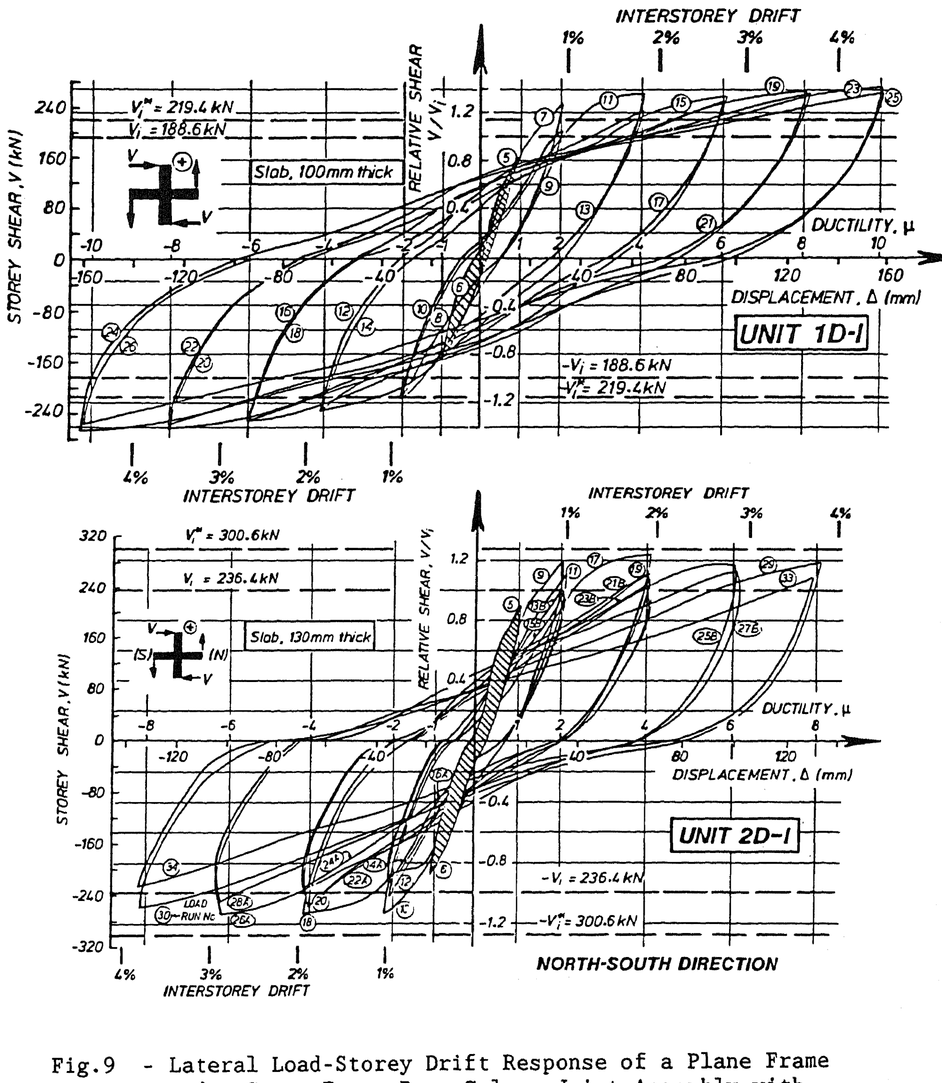

Fig.9 - Lateral Load-Storey Drift Response of a Plane Frame and a Space Frame Beam-Column Joint Assembly with Cast in Place Slabs [Ref.12].

properties of these units were achieved at the expense of using more transverse reinforcement and smaller diameter beam bars. Both measures reduced deformations orginating in the joint. Most researchers confirmed that, inspite of the perceived confining effects of transverse beams, the performance of space frame units was inferior to that of comparable plane frame assemblies. This is clearly demonstrated in Fig.9. While it was reassuring that storey drifts as much as 4% were achieved in most reported tests, it should be remembered that drifts in excess of 2% are not likely to be readily accommodated in high rise frames, because of the likely significant and detrimental influence of P-delta phenomena on both lateral load resistance and dynamic response.

The Contribution of Floor Slab One of the important findings in these studies was the universal confirmation of increased beam strength when the slab, acting as a flange, was in tension. It was also established that the participation of slab reinforcement, placed parallel to a beam, increased as imposed inelastic displacements were increased. However, this strength enhancement was different in one-way and two-way systems. Examples are shown in Fig.9 where values of column shear V_i were based on conservatively assumed [Ref.4] effective slab widths-typically 2 to 4 times the slab thickness beyond column edges, and where values of V_i^* quantify the theoretical participation of the entire slab width, both based on the measured yield strength of relevant bars. It is seen that in the plane frame unit (1D-I) full participation with strain hardening occurred, while, in spite of strain hardening, the full tensile strength of the flanges was not developed in the space frame (2D-I) unit. The increased strength of Unit 2D-I, in comparison with that of Unit 1D-I, resulted from the use of significantly larger amount of reinforcement in the two-way slab.

The Anchorage of Beam Bars A particularly difficult feature of experimental work is the separation of joint deformations due to joint shear and bond slip. The latter is commonly included in the beam deformations and therefore its magnitude is obscured. For the majority of the tests specimens, variations of steel strains or stresses along beam bars passing through the joint were recorded. This gave a good indication whether the bars performed or not in accordance with the usual design assumptions. In some cases compression strains could not be developed in such bars. The most significant influence of bond behaviour on overall performance is detected in changes of stiffness and energy dissipation. In some cases [Ref.8] stiffnesses relevant to elastic response, before any inelastic excursion, indicated yield displacement (i.e. $\mu = 1$) at approximately 1.5% drift. A suitable index to gauge the severity of anchorage conditions for beam bars in a joint [Refs.6,12] is the fictitious stress $X = d_b f_y / h_c$. This parameter, also referred to as "bond index", was chosen as a variable in one study [Ref.6]. There is a strong indication that stiffness and energy dissipation decrease with an increase of the index X, typical values of which, relevant to different tests, are shown in Table 1. The choice of an acceptable value for X is likely to depend on the desired elastic and hysteretic performance of a frame, rather than on the maximum level of resistance to be developed. The index does not necessarily express the severity of bond condition in every joint because it does not take into account the strength of the concrete and relieve from the fact that under certain conditions yield in tension and particularly

Table 1: Bond Index X (MPa)

Source	$X = d_b f_y / h_c$
Ref.6	11.7 to 20.5
Ref.7	23.0 to 34.0
Ref.8	19.1 to 22.8
Ref.9	16.4
Ref.10	10.8 to 16.8
Ref.11	14.9
Ref.12	11.3

in compression may never be developed at a column face. It has been suggested that the parameter in the form of $X/\sqrt{f'_c}$ may be more representative. The reported tests suggest that the limit $X \approx 11$(MPa)[Ref.4] may be unnecessarily conservative and that higher values may be adopted, in particular for frames with limited ductility demands.

The influence of bond on shear strength and on hysteretic response may be summarized as follows:

(a) When, by adopting a conservative value for the bond index, X, good bond conditions are provided, failure by diagonal tension across the joint core is enhanced. To prevent this, a significant amount of joint shear reinforcement is required. However, optimum hysteretic response can be attained. (Fig.9).

(b) With the use of large diameter beam bars and only nominal transverse reinforcement in the joint core, both of which appeal from the point of view of construction, joint deformations due to bond deterioration will be larger and frame stiffness will be reduced. Significant reduction of energy dissipation must therefore be expected.

SUGGESTED RESEARCH NEEDS

1. Possibilities for unconventional but viable solutions to reduce demands on beam-column joints, to ease or bypass problems arising from the dominating influence of shear and bond, could be explored.

2. Possible modifications, which might be required for the design of joints in non-rectilinear frames, should be studied.

3. An identification of features of the behaviour and design of joints, which are eccentric with respect to the axis of a column, is desirable.

4. The degree of acceptable relaxation of design requirements, in terms of both shear and bond response, could be quantified and refined for frames in which only limited demand for ductility is to be expected.

5. The tests reported have consistently shown that the stiffness of cracked beam-column assemblages, when subjected to a few cycles of reversed loading with intensities not approaching the yield strength of any member, is considerably less than that predicted with generally accepted and used analysis techniques. This anomaly should be resolved if code recommended drift limits for purposes of damage control, building separation or definition of displacement ductility, are to be used more realistically. The role of joint deformations in overall frame flexibility deserves special attention.

6. For convenience negligible or very small axial compression loads were applied to columns of the reported test units. Large axial compression on interior columns, resulting from gravity loading, may well permit further relaxation in the detailing of interior beam-column joints, situated at lower levels of medium to tall buildings.

7. Methods of strengthening or suitable modifications of existing and older structural frames, in which the design of joints was originally not considered, should be studied.

8. With the presentation of a wealth of data, the experimentally observed response of different joints was extensively documented. This will greatly assist in the design of beam-column joints, which have now been recognized

as critical regions of a structure requiring special attention. However, the sets of recommended design rules, most of which are entirely empirical, often read like recipes. This carries the danger of misuse by designers not sufficiently familiar with the underlying causes. It is for this reason that further studies, directed to the identification of mechanisms, the interplay of forces, the interaction of components, and in general better reasoned understanding of the behaviour of beam-column joint assemblies, should be encouraged. A set of rules without an appreciation of underlying first principles, is likely to stifle creativity and innovation in structural design.

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