ELASTIC BEAM-COLUMN JOINTS FOR DUCTILE FRAMES

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SUMMARY

Beam-column joints of ductile frames, when subjected to large earthquake induced inelastic displacements, may fail in shear or due to slippage of the flexural reinforcement passing through the joint core. Shear failures can be controlled by adequate joint core shear reinforcement. Bond failure due to penetration of yielding along beam bars into the joint core is more difficult to control. When potential plastic hinges in beams are deliberately removed from column faces, so that yielding of flexural reinforcement at the joint cannot occur, the elastic response of joints can be assured. This results in at least 50% reduction in demand for joint shear reinforcement and in the elimination of bond failure.

INTRODUCTION

It is now widely accepted that columns of earthquake resisting ductile frames should be stronger than the beams. Thereby storey sway mechanisms, which may impose excessive ductility demands on plastic hinges of columns during large earthquakes, may be avoided. Theoretical dynamic studies have indicated (1) that in ductile frames so designed the formation of plastic hinges at column-beam joints, even during very severe seismic excitations, can be restricted to the beams. To ensure that such desirable energy dissipating beam plastic hinges can be maintained during several cycles of reversed inelastic loading, beam-column joints must be suitably proportioned and reinforced so that they can sustain the largest loads trasmitted from the beams.

Column-beam joints, such as shown in Fig. 1, may fail in shear. The horizontal and vertical joint shear forces induced in the joint core, when beam hinges develop at the column faces, can be readily derived from first principles (2). When the joint shear reinforcement provided is insufficent, a corner-to-corner diagonal tension failure across the joint core results. With cyclic loading progressive reduction in both strength and stiffness is observed (3). Moreover, the intended plastic hinges in beams cannot be developed. Therefore the primary aim of the joint design must be to suppress a shear failure. This often necessitates a considerable amount of joint shear reinforcement, which may result in construction difficulties.

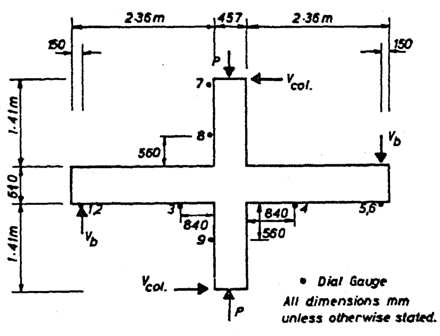

Fig. 1 - Dimensions and Loading of Test Specimens (5)

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The second and equally important source of possible failure in the joint core is deterioration or loss of bond. Very large bond forces need to be transferred, particularly at interior joints, from the beam bars to the surrounding concrete when plastic hinges form in the beams adjacent to the column faces. With progressive loading yielding of the beam bars in the plastic hinge regions spreads along the bars and penetrates into the joint core. This reduces the effective development length of the yielding bars (4). Eventually bars may completley slip through the joint, so that the yield strength of plastic hinges in beams adjacent to the joint can no longer be attained. Such bond failures are difficult to eliminate even in joints which are fully reinforced against shear failure. However, when small diameter bars are used in beams, slipping through the joint core can be sufficiently delayed (4).

When plastic hinges in beams are relocated from column faces, so that yielding of the beam flexural bars cannot occur at the column faces under the most severe earthquake loading, the anchorage of beam bars within the joint core is easier to achieve and additional benefits with respect to shear strength are also obtained. When the beam section at the column face remains elastic, the concrete compression zone in the beam will always contribute to the moment of resistance of that section. This concrete compression force in the beam combines with a similar force from the column and these two forces give rise to a diagonal compression strut across the joint core. This diagonal strut may transmit a considerable fraction of the total joint shear force to be resisted when the full strengths of the relocated beam plastic hinges are developed. Joint shear reinforcement is therefore required only for the remaining fraction of the total joint shear. The plastic hinge in the beam must be at a sufficient distance from the column face to ensure that the spread of yielding in the beam bars does not extend to the column face. The experimentally observed behaviour of two such interior beam-column joints is reported in this paper.

THE TEST SPECIMENS

The overall dimensions of the near full size test specimens, together with the applied load pattern, are shown in Fig. 1. Units B1 and B2 were made as near as possible identical with Units B12 and B13 which had been previously studied (4). In Units B12 and B13 beam plastic hinges at the column faces were responsible for causing the eventual slippage of beam bars through the joint core. The more important details of the Units B1 and B2, in which the joint was intended to remain elastic (5), and their conventional counterparts B12 and B13, are assembled in Table 1.

The average yield strengths of the bars used in the beams, columns and joints were, 293, 427 and 346 MPa respectively. The cylinder compressive strength of the concrete was approximately 30 MPa.

All specimens were cast without construction joints in the horizontal plane flat on the test floor. After placing the units in the test rig, controlled load or displacements were applied to the ends of the beams only, as shown in Fig. 1, while the columns were subjected to a constant axial compression force, P_{col}. Units B1 and B2 were initially tested by applying loading which was less than that required to cause yielding of the flexural reinforcement, thus simulating the actions at the joint of a unit with plastic hinges in the beams relocated away from the column faces. The prescribed maximum beam moment of 288 kNM for Units B1 and B2 was 86% of the theoretical moment to cause yield of the flexural reinforcement. This maximum moment was applied in 12 half cycles, i.e. 6 times in each direction, while the axial load on the column was held constant at the value showing in Table 1. Beam deflections and steel strains in various bars were monitored during the test. In Unit 2 additional 6 half cycles of load were applied while the axial load on the column, P_{col}, was reduced in two steps to $0.25\ f'_c\ A_g$. Then, in order to study the failure mechanism in these column-beam joints, not designed to resist shear forces resulting from adjacent plastic hinges in the beams, the maximum beam moment was increased till significant yielding of the flexural reinforcement occurred. As expected, at this stage of loading, failure of the joints reduced the capacity of the units.

By comparison the load in Units B12 and B13 was applied so that flexural yielding of beams occurred by progressively increasing imposed displacement ductilities, as measured by the tip deflection of the beams. The intention with those tests was to study the influence of the column-beam joint on the hysteretic response of the beam plastic hinges adjacent to the column faces.

A COMPARISON OF TEST RESULTS

The load-deflection relationship for Unit B12 is shown in Fig. 2. This may be considered to represent the satisfactory performance of a column-beam unit in which beam hinges develop at the column faces. It is seen that with increased inelastic displacements the maximum strength developed also increased. Up to a displacement ductility factor of $\mu = 4$ some reduction of stiffness was evident. However, for the intended purpose of seismic resistance the hysteretic response of the unit was judged to be quite acceptable. After two excursions up to $\mu = 6$, very significant loss of stiffness became evident. It was at this stage that yield penetration along the beam bars into the joint core resulted in slippage of these bars. This caused the significant reduction in energy dissipation. The deterioration of the joint is evident from the marked "pinching" in the hysteresis loops.

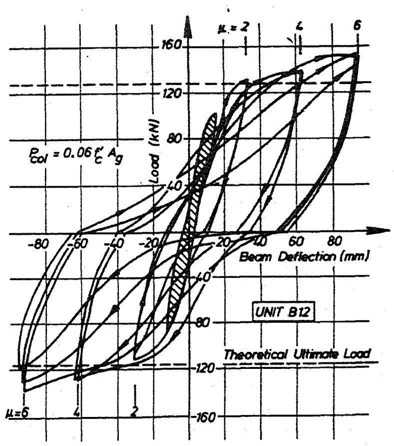

Fig. 2 - Load-Deflection Response of Unit B12 with Small Applied Axial Load (4)

By comparison the joint of Unit B1, which was intended to remain elastic, exhibited a very stable response during the first 12 half cycles of loading, while it sustained loads comparable with those of Unit B12. This is seen in Fig. 3 where the shaded area indicates the response for these elastic load cycles. It was concluded that the hysteretic response of such a unit would have been controlled predominantly by the response of the relocated plastic hinges in the beams, and that therefore the influence of the

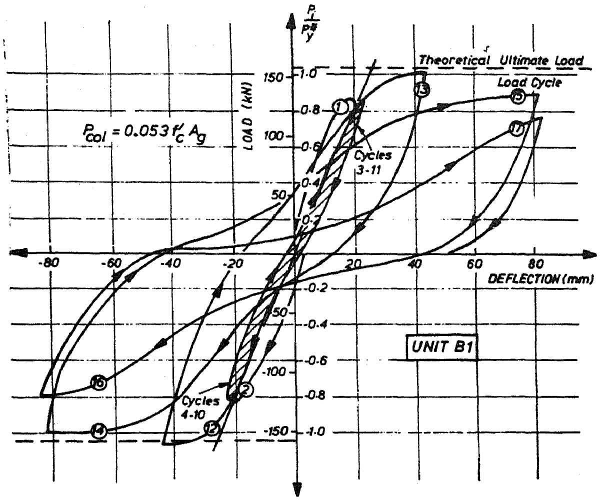

Fig. 3 - Load-Deflection Response of Unit B1 during Elastic and Subsequent Inelastic Loading (5)

elastic joint on inelastic response was negligible. This excellent joint response was achieved with horizontal joint reinforcement, approximately 50% of that used in Unit B12. (See Table 1.)

At the completion of the chosen performance test of Unit B1, as shown by the shaded area in Fig. 3, the displacement imposed on the beams were increased to correspond approximately with a displacement ductility factor of $\mu = 2$ and 4 respectively. While the single cycle to $\mu = 2$ was satisfactory, at $\mu = 4$ the theoretical strength of the unit could no longer be attained. The dramatic loss of strength and stiffness in the second cycle to $\mu = 4$ is evident in Fig. 3. This was due to the yielding of the horizontal joint shear reinforcement which resulted in a diagonal tension failure of the joint. It should be noted that with properly designed relocated plastic hinges in the beams, the load causing the shear failure of the joint in this test, could never have occurred during an earthquake. Therefore the second stage of this test, beyond cycle 12, demonstrated only that the joint shear reinforcement provided was inadequate for the case when yielding of the beam bars at the column faces can occur.

Axial compression on a column improves both the shear strength of and the anchorage conditions in a joint core. The response of Unit B13, illustrated in Fig. 4, is excellent. The hysteretic response shown is due to the plastic beam hinges at the column faces. The response is largely unaffected by distortions in the joint core. Significant stiffness degradation occurred subsequently in Unit B13 when the axial compression was reduced to $0.25\ f_c'\ A_g$. Eventually, as a result of 12 cycles of inelastic reversed loading, involving strain hardening of both the top and bottom beam reinforcement at the column faces and

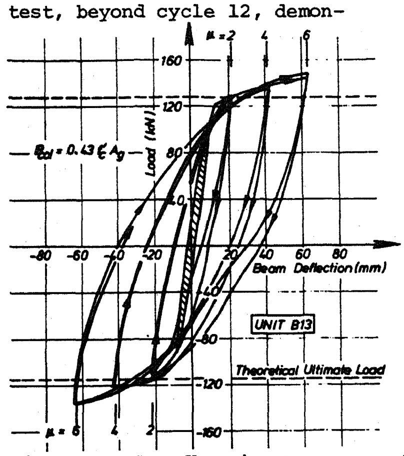

Fig. 4 - Load-Deflection Response of Unit B13 with Large Applied Axial Load (4)

consequent yield penetration into the joint core, the beam bars slipped through the joint.

The "elastic" joint core of Unit B2, being subject to the same axial compression as that of Unit B13, exhibited negligible deterioration during the first 12 semicycles of loading. This is shown by the shaded area in Fig. 5. If plastic hinges, removed from the beam faces, would have been present in Unit B2, a response very similar to that obtained for Unit B13, shown in Fig. 4, would have resulted. The joint shear reinforcement used in Unit B13 was, however, 5,7 times as much as that provided in Unit B2. Subsequent loading of Unit B2, with axial load reduced in two steps to 0.25 $f'_c A_g$, resulted only in a small reduction of stiffness in load cycle 18.

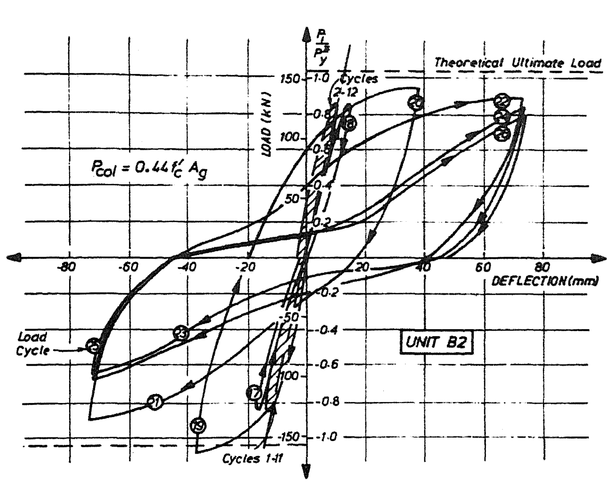

Fig. 5 - Load-Deflection Response of Unit B2 during Elastic and Subsequent Inelastic Loading (5)

After this satisfactory performance test of Unit B2, the beam loads were increased into the yield range while the column load was maintained at 0.44 $f'_c A_g$. As Fig. 5 shows, rapid loss of stiffness and strength resulted. It is to be noted that only once, in cycle 19, was the theoretical flexural strength of the unit attained. On reversal of the load in cycle 20 the contribution of the concrete strut in the joint core to shear resistance diminished. The horizontal stirrup ties provided were not capable of resisting their share of joint shear. Consequently a shear failure of the joint core resulted. When numerous steep wide diagonal cracks in both directions appeared, the core concrete was unable to sustain the column load. The concrete in the joint core crushed and all column bars across the joint yielded in compression. Thus the joint failure destroyed the load carrying capacity of the column.

ANCHORAGE OF BEAM BARS IN THE JOINT CORE

Strain measurements along beam bars passing through the joint indicated that during the elastic response up to maximum load, approximately uniform bond stresses developed along the bars in the joint core. This may be seen in Fig. 6, where the stress distribution along top beam bars are plotted. At the end of the 11th elastic load cycle, approximately at a ductility of $\mu = 1$, the concrete compression zone of the beam (at the left hand side of the joint) contributed considerably to the shear strength of the joint because steel com-

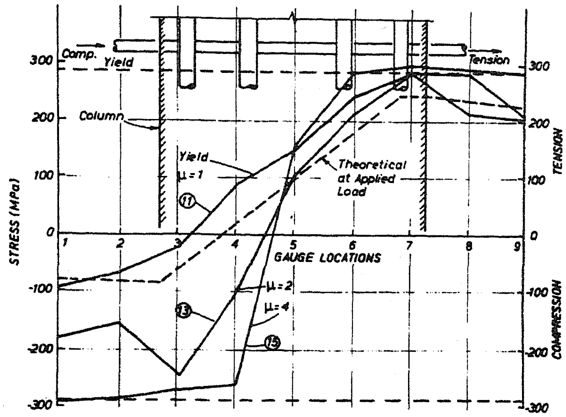

Fig. 6 - Stresses Along the Top Beam Bars within the Joint of Unit B1 (5)

pression stresses were only of the order of 0.3 f_y. However, after the 12th cycle, when tensile yielding was imposed on these bars, their contribution to flexural compression increased, and by cycle 15 the contribution of the concrete to flexural resistance of the beam at the column face, and hence its contribution to joint shear strength, had vanished. Joint shear failure must then result. Fig. 6 also shows the extent of yield penetration into the joint area at this stage, and the consequent dramatic increase in bond stresses in the centre of the core.

RESPONSE OF STIRRUP TIES IN THE JOINT

The previously described behaviour is also verified by the response of the stirrup-ties in the joint cores. Strains measured along all four sets of the joint shear reinforcement for Unit B1 are shown in Fig. 7. Only the average strains for the three locations along the ties are shown. There was only a small and gradual increase in stirrup strains during the first 12 "elastic" cycles of loading, as is indicated by the shaded area in Fig. 7. This was followed by an approximate 100% increase in strains when yielding of the beams commenced at $\mu = 2$. The onset of failure of the joint in diagonal tension is evident in the 14th cycle when all stirrups have yielded. By comparison only a few stirrups had yielded in Unit B12 when $\mu = 6$ was attained, and the increase in strain beyond yield was not great.

CONCLUSIONS

1. The elastic response to cyclic reversed loading of interior beam-column joints will ensure the maximum participation in shear resistance of the concrete of the joint core. To achieve this, it is necessary to ensure that yielding of the beam flexural bars at column faces cannot occur during the inelastic response of a frame due to earthquake motions.

2. If the beam flexural steel cannot yield at the column face,

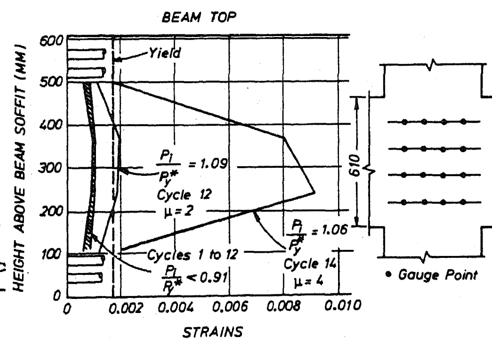

Fig. 7 - Joint Stirrup-Tie Tension Strain Distribution in Unit B1 (5)

the contribution of the diagonal concrete strut in the joint core to shear resistance is maintained, and greatly reduced joint shear reinforcement is required. When the axial compression on the column is significant, confinement of concrete and other requirements are likely to necessitate the use of more joint reinforcement than considerations of joint shear strength.

3. When yield penetration of the flexural steel into the joint core cannot occur, bond deterioration with cyclic loading will be insignificant. Consequently larger diameter beam bars may be used than in the case when beam plastic hinges develop at column faces.

4. The overall inelastic response of units with elastic joints, such as those studied, will be governed solely by the response of the plastic hinges in the beams located away from the column faces. Provided that sliding shear in such beam plastic hinges is controlled (6), a response similar to that shown in Fig. 4 is to be expected.

5. The relocation of plastic hinges, away from column faces, requires more flexural reinforcement at column faces than in conventional design. This is because the maximum stresses in the beam bars at these sections must not exceed the guaranteed yield strength when during an earthquake the overstrength (inclusive strain hardening) of the relocated plastic hinges might be developed.

6. The relocation of plastic hinges in beams requires careful detailing of the reinforcement (6). A relocated plastic hinge should be as close to the adjacent column face as practicable, otherwise plastic hinge rotations might need to become unnecessarily large. The anchorage of curtailed beam bars must be such that yield will not spread from the critical section of the plastic hinge into the joint region.

7. Only first principles, associated with the model of a diagonal compression strut and two orthogonal tension fields, are required to derive the amount of shear reinforcement necessary in beam-column joint cores to ensure that elastic joints, subjected to many cycles of reversed load, will not fail in shear (2).

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TABLE 1. PROPERTIES OF TEST UNITS

Designation of Unit	M_y (kNm) (1)	M_u (kNm) (2)	Applied Moments (kNm) (3)	A_{jh} (mm^2) (4)	Number of Sets (5)	Spacing of Sets (mm) (6)	Diameter of Ties (mm)	$P_{col}/f'_c A_g$ (7)
B1	334	347	288	2027	4	120	12.7	0.053
B12^x	237	256	321	4054	8	55	12.7	0.060
B2	334	347	288	531	4	126	6.5	0.440
B13^x	237	256	314	3040	6	75	12.7	0.430

x These units were tested previously (4).
1. Theoretical moment causing yielding of the flexural beam bars.
2. Theoretical flexural strength of beam section at column face.
3. The maximum applied moments occurred in Units B12 and B13 at displacement ductilities of $\mu = 6$, when strain hardening of the flexural bars developed. For units B1 and B2 it was decided to use the average of the theoretical and maximum observed loads obtained for Unit B12, i.e. 288 kNm, so that the corresponding joint forces in all specimens were of nearly the same magnitudes.
4. Total area of horizontal joint shear reinforcement consisting of 4 legged stirrup-ties and placed in the joint core between the top and bottom layer of beam flexural reinforcement.
5. One tie set consisted of 4 horizontal transverse legs in each direction of the column section.
6. The vertical spacing of horizontal stirrup-tie sets in the joint core.
7. The column load, P_{col}, is expressed in terms of the concrete strength and the gross concrete area of the column section A_g. The axial loads in Units B2 and B13 were subsequently reduced to $0.25\ f'_c\ A_g$ during the tests.