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ENERGY ABSORPTION IN WELDED BEAM COLUMN CONNECTIONS

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Welded beam-to-column moment resistant connections have been tested under gradually increasing static and pulsating loads to determine the energy absorption characteristics of these connections. Six specimens have been tested-3 each under static and pulsating loads. Three different sizes of beams have been used but the column section has been kept the same in all the specimens.

Beam to column connections were made using butt welds, backing plates and side cleat angles.

An optical technique was used for the measurement of rotations for the beam and column. In four specimens the full plastic moment of the beam was transferred across the connections. Introduction of diagonal shear stiffeners in the columns reduced the rotational capacity of the specimens considerably and consequently the energy absorption. Shear failure in the column web and weld fracture at the column face were the common observed modes of failure of the specimens.

Keyword : energy absorption, beam-column, connections

NOTATION

B : Width of flange
D : Depth of section
S : Plastic modulus of section
t_f : thickness of flange
t_w : thickness of web
θ : rotation of beam in radians
Φ : rotation of column in radians
β : Damping factor

1. INTRODUCTION

In the analysis of steel framed buildings, the beam-column joints are usually assumed to be rigid and the effect of the type, size and strength of the joint on the static and dynamic behaviour of the frame is generally neglected. Earlier investigations[1),2)] into the behaviour of these connections have revealed that joint deformations are important and improper design of connections could lead to a significant drop in energy absorption capacities of frames under dynamic loadings.

In steel frames, connections can be either of bolted or welded type. In welded beam-to-column connections, the beam is butt welded to a column flange and the forces from the beam are transferred to the connected column through the welds. The mechanism of transfer of these forces from the beam to the

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column is complex and an accurate analysis of the connection is difficult.

In the past, tests have been carried out on beam-to-column moment resistant connections by Parfitt and Chen[3] to determine the behaviour of, and to develop a design method for these connections. Dawe and Kulak[4] studied welded connections subjected to the combined action of moment and shear to predict the ultimate load carrying capacities of eccentric weld groups, based on an empirical relation developed by Butler and Kulak[5] for the load deformation response of weld elements under different directions of loading.

Butler, Pal and Kulak[6] investigated the load-deformation behaviour of eccentrically loaded welded connections and developed a theoretical method for predicting the ultimate loads.

In 1978, the Australian Institute of Steel Construction published a manual[7] for the 'Design of Structural Connections' for Australian universal sections, suggesting procedures to be adopted for the design of connections both by elastic and plastic approaches. By using the design criteria available in this manual, the strength requirements of connections are met, but the need for ductility and moment distribution under the serviceability and limit states has not been emphasised. The need for ductility becomes important when a plastic method is adopted for design.

It is to be noted that according to the Australian design criteria, components of connections are sized and evaluated independently and the interactive effect of different components is ignored-thus introducing an empirical aspect to the design.

A literature review of the energy absorption capacities of welded beam-to-column connections indicated very limited information. The property of damping in composite structures has been studied by Jacobsen[8] and in friction grip bolted joints by Vitteleschi and Schmidt[9].

In 1982, a test program was initiated at the Papua New Guinea University of Technology to study the energy absorption characteristics of bolted and welded connections under static, pulsating and cyclic loads to determine the damping coefficients for steel structures and to compare the actual coefficients with the damping coefficients specified in earthquake loading codes.

The phenomenon of energy absorption is important for areas experiencing frequent earthquakes, since it is desirable for load carrying structures to dissipate earthquake energy quickly.

Damping in bolted end plate beam column connections has been studied by the authors[10)~12)] under static and pulsating loads and the results reported elsewhere.

2. DESCRIPTION OF SPECIMENTS

To study the effect of beam size on the damping capacity of beam column connections, six specimens were tested. Three were tested under gradually increasing 'static' loads, while the other three were subjected to pulsating loads of low frequencies.

All specimens were made using BHP sections conforming to AS-1204[13] with a nominal yield stress of 250 N/mm^2. The measured dimensions of the sections are given in Table 1.

Table 1 Dimensions and Plastic modulii of Sections.

Section	D	B	t_f	t_w	S X 1000.0	Calculated M_p	M_p
	(mm)	(mm)	(mm)	(mm)	Without backing plate (mm^3)	At the edge of backing plate (kN-m)	At the face of column (kN-m)
200 UC 46.2	206.00	204.00	10.92	8.02	502.57	-	-
250 UB 31.4	252.58	146.68	8.54	6.30	393.05	117.52	193.91
200 UB 25.4	205.37	133.73	7.64	6.11	257.12	76.88	132.49
152x76x17.9	152.03	75.28	9.62	5.86	132.01	39.47	59.28

N.B. Fully plastic moments have been calculated using a yield stress of 299 MPa. See Para 2.1.

The column size used in all the specimens was 200 UC 46.2 kg/m with British equivalent of 203×203 UC 46. Three different beam sections were used i.e. 200 UB 25.4 kg/m, 250 UB 31.4 kg/m and a tapered flange beam 152×76×17.9 kg/m with British equivalents of 203×133 UB 25, 254×146 UB 31 and a joist 152×76×17.86 respectively. Two test specimens were made using each beam section. The plastic method of design was used for calculations and other dimensional details were fixed according to the Australian 'Design of Structural Connections, Standardised Connection Manual Part B[7]. A sketch of the beam column connection used for testing is shown in Fig. 1.

In all specimens, the beam was butt welded to a column flange using E 48 welding electrodes. (Electrodes with nominal tensile strength of 480 N/mm^2) Beam flanges were initially surface ground as shown in Fig. 2. A 10 mm thick backing plate was used under each flange after coping the beam web at each end.

(1) Determination of yield stress

To have an accurate assessment of the yield stress of the beams and column material, 21 tensile specimens were subjected to axial tensile load, Four specimens were obtained from the flanges of each of 200 UB 25.4, 200 UC 46.2 and 250 UB 31.4 and another 3 each were obtained from the webs of these sections. An average yield stress for the flange material was observed to be 283±12 N/mm^2 while the web material had a higher yield stress of 315±15 N/mm^2 compared with the nominal yield stress of 250 N/mm^2. The combined average yield stress of the beams and column material is 299 N/mm^2.

(2) Stiffeners

a) Beam stiffeners

Two full depth stiffeners were welded between the beam flanges symmetric to the load application point on each side of the beam.

b) Column stiffeners

Two 6 mm full depth stifferers were transversely welded between the column flanges and in line with the beam flanges. (Fig. 1). In addition, for specimens W 3 and W 4, a full depth 10 mm thick shear stiffeners was diagonally welded to the column web on each side.

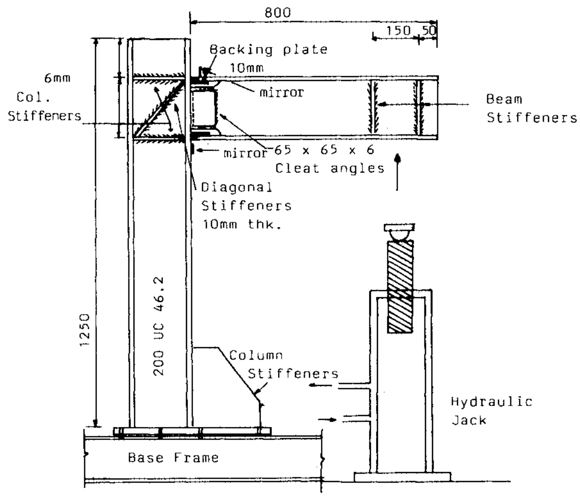

Fig. 1 Test Specimen and Loading Arrangement.

3. LOADING OF SPECIMENS

For loading, beam-column joints were mounted on a base frame which was designed to suit the foundation bolts in the test floor. The base of the beam-column connections was fastened to the base frame using 10-20 mm diameter bolts. The base of the column was made relatively stiff to avoid excessive deformations of the base. The load was applied to the connections by a 20 t (196 kN) hydraulic jack resting on the floor and operated by a

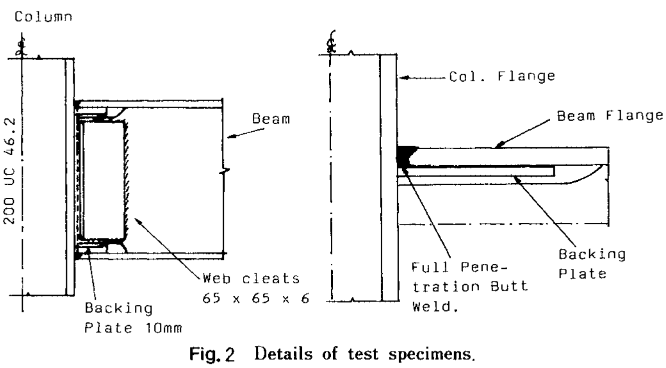

Fig. 2 Details of test specimens.

Mohr & Federhaff hydraulic power source.

(1) 'Static' load tests

Three specimens tested under static conditions were loaded up to failure in discrete increments of load. The connection was assumed to have failed either when large displacements were observed for a constant load or when the load maintaining device on the Mohr & Federhaff machine had difficulty in maintaining the load. At each load, a complete cycle of loading and unloading was carried out. An initial cycle of loading corresponding to the maximum elastic moment (except for specimen W 4) of the beam was carried out before incremental measurements were taken.

(2) Pulsating load tests

Three specimens were subjected to pulsating loads of 1 500 cycles at each step with a gradually increasing maximum but a fixed minimum for each beam as shown in Fig. 3 using a load alternator coupled to the main stand of the Mohr & Federhaff machine. To study the effect of energy absorption with the number of load cycles, the load alternator was stopped after every 500 cycles and a set of observations was recorded for loading and unloading of the specimen. In all the tests, the frequency of load application was kept at 0.6 Hz or 1.67 sec./cycle.

The transition from the lower load to the upper load was smooth and the load was maintained approximately for equal duration at both the upper and lower loads.

4. MEASUREMENTS

To measure the energy absorbed by the joint, it was necessary to measure the rotation of beam 'θ' and rotation of column 'Φ'. The area enclosed within the loop of moment (calculated on the face of column) versus $(\theta-\Phi)$ for a complete cycle of loading and unloading gives a measure of energy absorbed by the joint.

To measure the rotations of beam and column, an attempt was earlier made for the automatic measurement and recording of hysteresis loops by using 4 linear voltage displacement transducers —2 each mounted on beam and column. On loading a test specimen, out of plane movement of the beam-column connection was also observed which despite all efforts could not be avoided. It was therefore necessary to record both the in and out-of-plane rotations of the joint. An optical technique was adopted by using a pair of ordinary plain mirrors and theodolites. One mirror each was mounted on the beam and column close to the connection. (Fig. 1) Graph papers were mounted on vertical boards to serve as sighting targets. Reflected images of these graph papers observed in the mirrors were viewed using theodolites. A schematic arrangement used for the measurement of rotations is shown in Fig. 4. Under load, it was possible to read on graph papers, the magnified rotations of beam and column, both in and out, of the plane of the joint. By using a pair of theodolities, it was possible to measure the rotations of beam and column

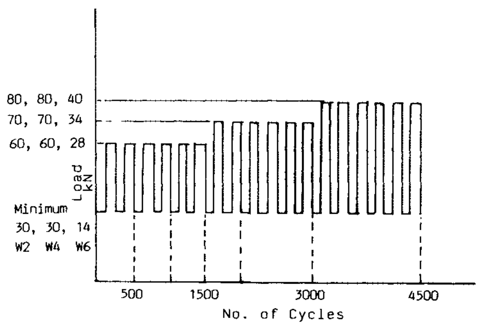

Fig. 3 Loading of specimens under Pulsating Loads.

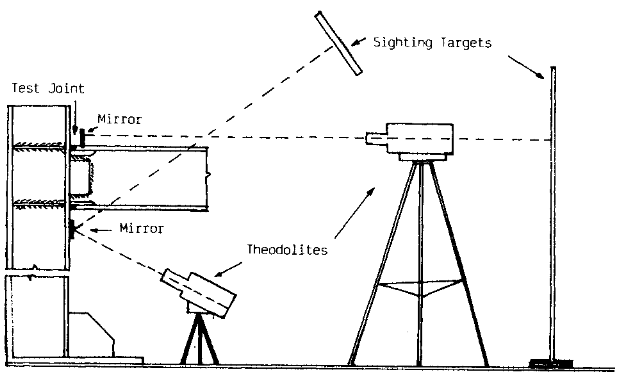

Fig. 4 Arrangement for Rotations Measurement.

independently. This technique though involving some computation proved to be effective and simple for rotation measurements.

Rotations of the column base were also recorded using a vertical dial gauge for every observation and were subtracted from beam/column rotations.

5. OBSERVATIONS

A comparative table of the six specimens tested is given below (Table 2).

(1) Static load tests

Specimen W 1 tested under static loads transferred 80.4 kN-m moment (calculated on the face of the column) across the connection - an increase of 4.6 % over the calculated plastic moment of the beam. (It should be noted that the plastic moment capacity of the beams has been calculated using the observed (average) yield stress of 299 N/mm^2 together with the plastic modulii calculated at the edge of the backing plate using the section properties given in Table 1.) At failure, large deformations were observed in the area between the column stiffeners (shear zone) and a typical deformed column section is shown in Fig. 5. No deformation in the beam was observed however. Maximum in-plane rotation of the joint at failure was 61.9×10−3 radians. At failure, the column web was subjected to 287 kN shear force against the maximum calculated value of 285 kN-using a yield stress of $F_y/\sqrt{3}$. where F_y=299 N/mm^2.

Specimen W 3 transferred 93.8 kN-m moment (calculated on the face of the column) across the connection as compared to the calculated plastic moment of 117.5 kN-m. This specimen was initially tested

Table 2 Comparison of Specimen Rotational Capacity

Specimen No.	W1	W2	W3	W4	W5	W6
Beam Size	200UB 25.4	200UB 25.4	250UB 31.4	250UB 31.4	152x76x17.9	152x76x17.9
Nominal & Calculated moment capacity (kN-m)	65.0 76.88	65.0 76.88	99.0 117.52	99.0 117.52	33.25 39.47	33.25 39.47
Column Size its Nominal flange thickness	200UC 46.2 11.0	200UC 46.2 11.0	200UC 46.2 11.0	200UC 46.2 11.0	200UC 46.2 11.0	200UC 46.2 11.0
Nature of load applied	Static	Pulsating	Static	Pulsating	Static	Pulsating
Beam Stiffeners thickness (mm)	10.0	10.0	10.0	10.0	10.0	10.0
Column Stiffeners & their separation & their	6.0 200.0	6.0 200.0	6.0 200.0	6.0 200.0	6.0 152.0	6.0 200.0
Shear stiffeners (mm)	-	-	10.0	10.0	-	-
Max. S.F. (kN)	120.0	116.0	140.0	150.0	90.0	80.0
Max. Moment measured at column face (kN-m)	80.4	77.72	93.8	100.5	60.3	53.6
Max (Θ-Φ)x10-3 rotation in plane of joint	24.1	16.6	42.0	7.9	30.1	19.7
Max (Θ-Φ)x10-3 rotation out-of-plane of joint	2.26	1.0	2.35	1.50	8.1	2.3

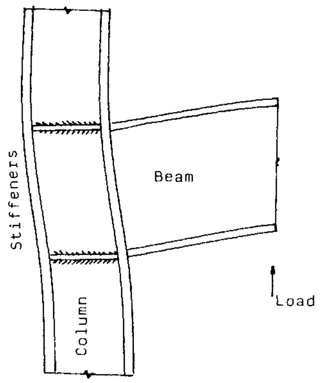

Fig. 5 Typical deformed shape of test joint.

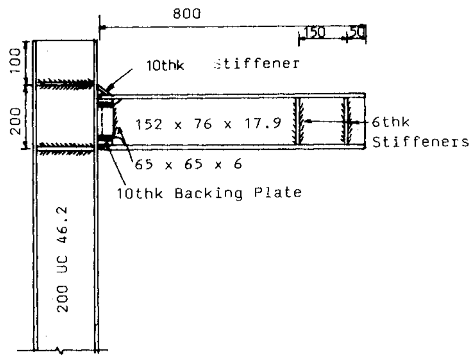

Fig. 6 Details of specimen W 5.

without shear stiffeners upto 80.4 kN-m moment, when the test had to be stopped because excessive deformations in the column were observed and the Mohr & Federhaff machine could not maintain the load. Yield of the coiumn web had initiated the failure and maximum rotation in and out of the planes of the connection were $42.1 \times 10-3$ and $2.35 \times 10-3$ radians respectively. Two 100 mm shear stiffeners were then welded to the column-one on each side. (Fig. 1) With additional stiffeners, it was possible to load the specimen further and an additional 13.4 kN-m moment could be transferred across the connection when failure of the specimen occurred with the yielding in column flanges. However, the maximum moment transferred was still 20.2 % lower than the calculated plastic moment of the beam. Again no visual deformation of the beam section was observed. By welding two-10 mm full depth shear stiffeners, to the column, the shear capacity of the column had increased to 403 kN, but the failure occurred when the column web had a maximum shear force of 384 kN.

It should be noted that the shear deformations of the joint panel contributes significantly to the overall ductility of the frame, but it should also be realised that excessive deformations in this area could produce a rapid change in column curvature.

The behaviour of the panel zone has been extensively studied[14)~16)] and the methods of reinforcing this area either with doubler plates or plates parallel to column web are reasonably well known.

Specimen W 5, tested under static loads had a slightly different configuration. Fig. 6. The size of the shear box was made bigger and was found to transfer 55.8 kN-m (calculated at the edge of the backing plate) moment across the connection-an increase of 41.4 % over its calculated plastic moment. The failure of this joint occurred by the yielding of the beam compression flange and lateral bending of the beam. The yielding of the beam flange occurred at the junction of the backing plate and the flange, because beyond this point, the backing plates, tend to increase the fully plastic moment of the beam from 39.5 kNm to 59.9 kNm.

(2) Pulsating load tests

Under pulsating loads, specimen W 2 transferred 77.7 kN-m moment across the connection -an increase of only 1 % over the plastic moment of the beam calculated at the edge of the backing plate. Failure of the specimen initiated with a crack in the beam to column tension flange butt weld, which opened and closed under pulsating load, as it propagated. After 980 cycles, the crack traversed the entire length of the tension flange and complete failure of the joint was observed as seen in Fig. 7. Examination of the ruptured surfaces revealed a ductile failure of the joint.

In the case of a specimen W 4, the failure was again initiated by a crack in the beam to column tension flange weld. The failure was violent, with loud noise and tore the column flange and cleat angle welds (Fig. 8). The joint at the time of failure was carrying only 52 % (100.5 kNm) of the fully plastic moment of the beam (193.91 kN. m) calculated at the face of the column with the inclusion of backing plate in the

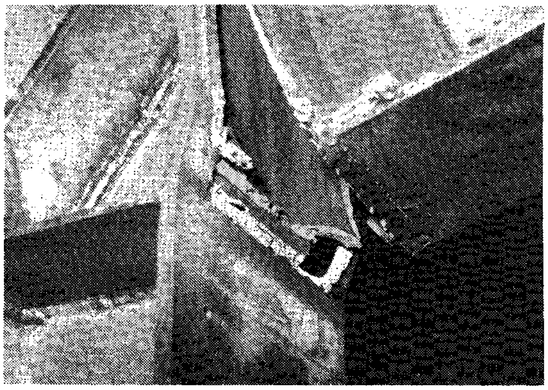

Fig. 7 Failure of Specimen W 2.

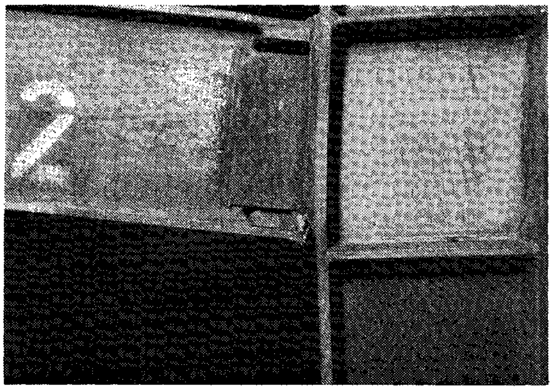

Fig. 8 Failure of Specimen W 4.

section. (Ref Table 1). It is observed that shear stiffeners in the columns greatly reduced the ductility of the specimen. Ruptured surfaces of the specimen were examined and the failure was partially brittle and partially ductile. In this specimen, joint rotations were much lower than in specimen W 3 tested initially without stiffeners.

Specimen W 6, tested under pulsating loads with column stiffeners in line with the beam flanges, transferred 49.6 kN-m (measured at the edge of the backing plate) moment across the connection compared with the calculated beam plastic moment of 39.5 kN-m. The failure of the specimen was due to simultaneous yielding of the beam compression flange and a crack in the weld adjacent to the backing plate.

6. HYSTERESIS LOOPS

Plots of moment versus ($\theta - \Phi$) for both the in-plane and out-of-plane rotations for each load increment were obtained separately for all specimens. For specimens tested under gradually increasing static loads, the hysteresis loops were plotted with respect to the total accumulated rotations, while for specimens tested under pulsating loads, the loops were drawn with reference to the origin of the axis system. A set of hysteresis loops obtained for specimen W 1 tested under static loads is shown in Fig. 9.

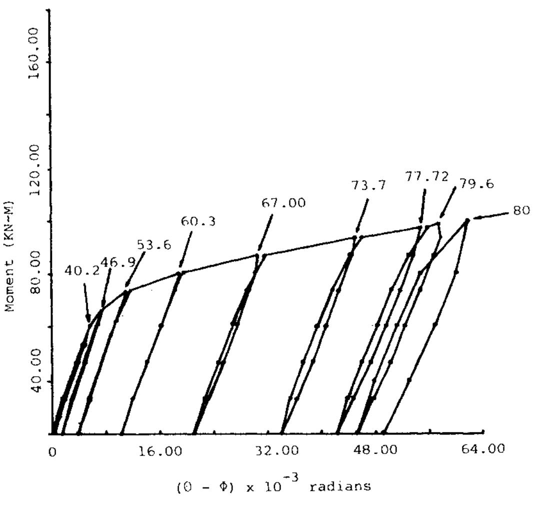

Fig. 9(a) Hysteresis Loops for Specimen W 1 for in-plan rotations.

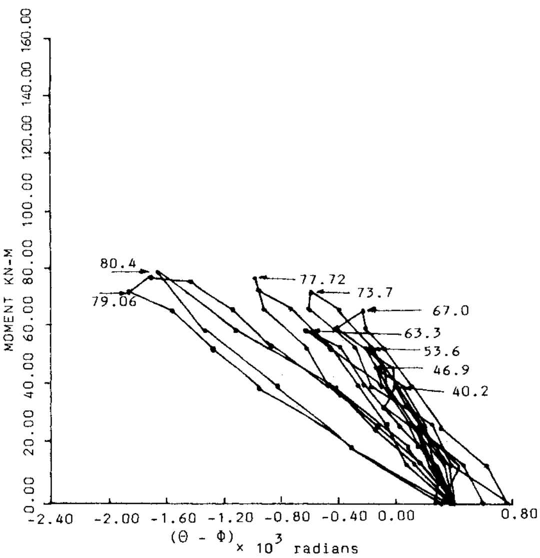

Fig. 9(b) Hysteresis Loops for Specimen W 1 for out-of-plane rotations.

The area enclosed within an hysteresis loop represents the energy absorbed for a particular cycle and damping is considered proportional to this energy absorbed. The measurements of areas were made on bigger print-out (obtained separately for each load cycle) to give a greater accuracy.

Damping has been defined by Jacobson in terms of 'Damping factor' based on the ratio of energy absorbed per cycle to the work capacity of the joint Fig. 10.

The energy absorbed per cycle 'P' is represented by the area OLABO and the work capacity 'Q' is represented by area OLACO. For specimens tested under static loads, the work capacity of the joint at any particular load increment has been calculated on the basis of total accumulated rotations, while for specimens tested under pulsating loads, the work capacity does not include overall rotations. In accordance with established practice, an approximation can be made by assuming the linear skeleton curve OA instead of the actual one. The damping factor is taken as :

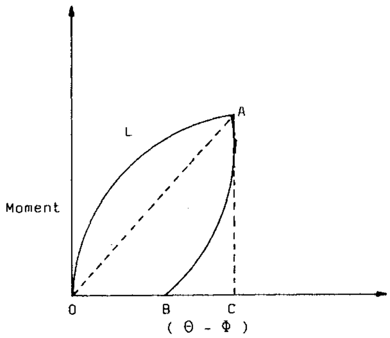

Fig. 10 Hysteresis Loops and Evaluation of Damping Factor.

$$\beta=\frac{1}{2\pi}\frac{(P)}{(Q)}$$

It may however be noted that damping factors calculated using the above relation will depend entirely on the ratio of the energy absorbed by the joint to work capacity. However different definitions can be assigned to the 'work capacity' of the joint. It should however be borne in mind that the above expression for damping has been used in mechanical engineering structures subjected to vibratory loads of relatively high frequency. As there appears to be very limited published work for civil engineering structures in this field, the above expression was adopted for the evaluation of damping factors.

The damping factors computed using the above relation have an upper and a lower limit depending on the magnitude of the energy absorbed by the joint and its work capacity. If the energy absorbed is very small as compared to the work capacity, (case with small plastic deformations) the damping factor is also small and it would approach zero, as the energy absorbed by the joint reduces. However, in the case of large plastic deformations, the total energy absorbed by the joint will approximately equal to the work capacity and the damping factor would then approach a limiting value of $1/2\pi$ or 0. 159. It may be noted, that, under large plastic deformations, the assumption of a linear skeleton curve OA instead of the actual one (Fig. 9) may be inappropriate.

For static loads, the hysteresis loops were drawn for each load increment whereas for pulsating loads, each load increment had 4 sets of hysteresis loops i. e. before the application of pulsating load and after 500, 1 000, and 1 500 cycles. The area enclosed within each loop was measured using a planimeter. The energy absorbed was measured for in-plane rotations of the joint. Plots showing the variation of damping factor against the percentage of calculated beam plastic moment (calculated on the face of the column) are shown in Fig. 11 (a) for specimens tested under static loads and Fig. 11 (b) for pulsating loads.

Under static load it will be seen that, initially, the damping factors increased as the moment on the joints increased. After a distinct peak in all the specimens, the damping factors decrease. For bolted endplate beam-column connections tested by the authors, similar behaviour was observed. The damping factors were seen to increase as the plastic moment of the beam was approached and thereafter they dropped.

Specimen W 3, tested with and without shear stifferers indicated an abrupt change in the damping factor when shear stiffeners were introduced in the column-as expected the connection stiffness increased and its rotational capacity decreased.

Under pulsating loads, specimen W 2 approached 0. 12 as the damping factor. It may be noted that a bolted connection with the same size of beam and column gave a similar maximum value. Specimen W 4

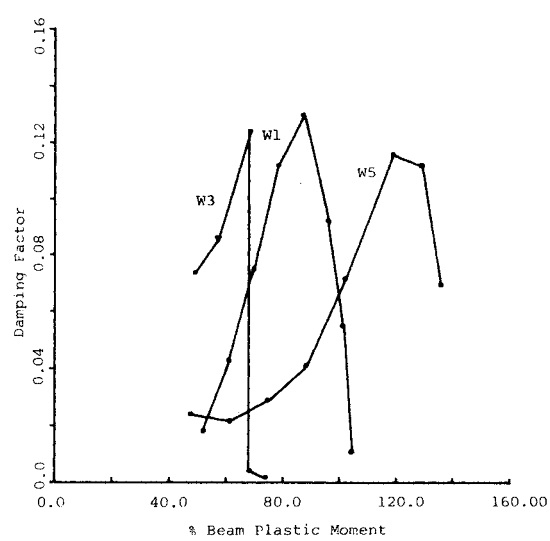

Fig. 11 (a) Variation of Damping Factor with % Beam Plastic Moment. (Specimens tested under Static Loads)

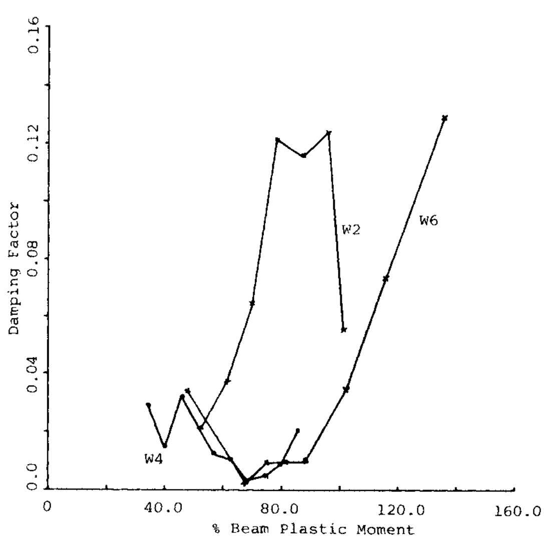

Fig. 11 (b) Variation of Damping Factor with % Beam Plastic Moment (Specimens tested under Pulsating Loads)

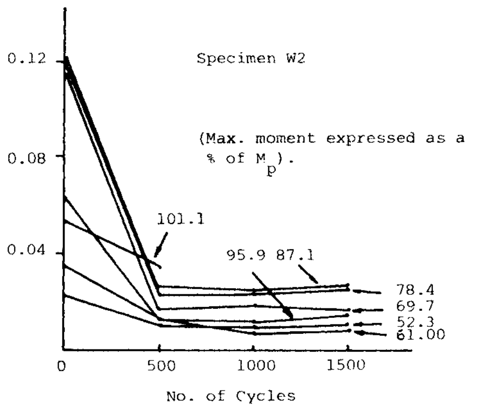

Fig. 12 (a) Variation of Damping Factor with No. of load cycles.

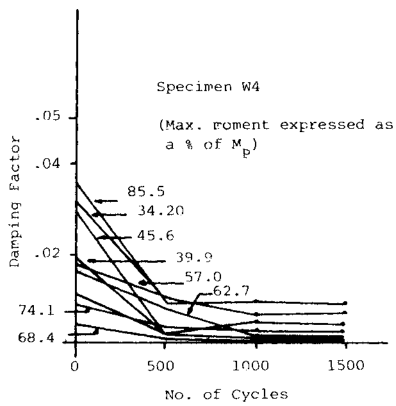

Fig. 12 (b) Variation of Damping Factor with No. of load cycles.

revealed a very low value for the damping factor because of the high stiffness of the joint.

Plots of damping factor versus number of load cycles for specimens W 2, W 4 and W 6 were obtained and are shown in Figs. 12 (a), 12 (b) and 12 (c) respectively.

It will be observed that the damping factor drops abruptly during the first 500 cycles at a particular load and thereafter remains practically constant with minor variations. This fact is also reflected in the hysteresis loops, which become relatively narrow after the first 500 cycles possibly on account of strain hardening.

The decision to load the specimens for 1 500

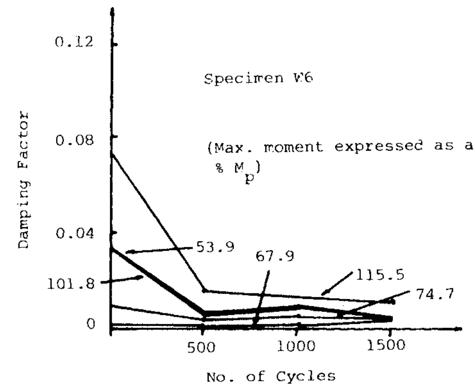

Fig. 12 (c) Variation of Damping Factor with No. of load cycles.

cycles at each load increment was taken after the trial testing of beam-column joint under pulsating load. The results had revealed that the energy absorbed per cycle had approached a stable value around 1 500 cycles.

7. CONCLUSIONS

Moment-rotation characteristics of welded beam-column connections have been obtained experimentally both under gradually increasing 'static' and pulsating loads. Specimens W 1, W 3 and W 5 tested under static loads failed in two distinct modes. Specimens W 1 and W 3 failed when the shear capacity of the column web was reached, while specimen W 5 failed with buckling and lateral bending of the beam compression flange adjacent to the edge of the backing plate.

For specimens W 2 and W 4-tested under pulsating loads, the failure was caused by a fracture in the weld at the column face. Specimen W 6, also tested under pulsating loads, failed with a yield in the beam compression flange and a crack in the weld adjacent to the backing plate.

The damping factors were seen to increase as the magnitude of the moment increased. Under static loads, the damping factors initially increased with the increasing loads and thereafter dropped. Under pulsating loads, a similar trend was observed for specimen W 2 only, while for specimens W 4 and W 6, the damping factors increased with increasing moment on the joint.

The diagonal shear stiffeners in specimens W 3 and W 4 reduced the rotational capacity of the shear panel -area between the column stiffeners. However, the stiffeners increased the load carrying capacity of the joint.

Variation of the damping factor with the number of load cycles revealed that the damping factors drop abruptly during the first 500 cycles and thereafter remains relatively constant with only minor variations. With load cycles, the hysteresis loops become thinner, showing the effect of strain hardening. Similar trend in the variation of damping factor with the number of load cycles was also observed for bolted connections tested under pulsating loads.

There appears to be little published work showing the effect of damping capacity on the shear force/bending moment ratio. It is proposed to determine the behaviour of connections subjected to similiar moments but varying shear force.

Finally, it is not easy to use directly in design the knowledge of ductility and energy absorption capacity of beams, columns and connections. Nevertheless, structures should be designed for their maximum possible strength and energy absorption capacity to obtain their best response under seismic loads.

8. ACKNOWLEDGEMENTS

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