

## CYCLIC LOADING TESTS ON STEEL PORTAL FRAME KNEE JOINTS

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### SYNOPSIS

The results of cyclic loading tests on three full-size portal frame knee joints are presented. All members complied with the flange and web slenderness limits for plastic design (AS 1250-1981) but were the lightest sections for their respective depths. Large decreases in load capacity were observed when either lateral-torsional or combined local flange and web buckling occurred. Lateral restraint forces were measured and were found to exceed the minimum ultimate strength values specified by the design code rules. One fully welded connection and two bolted connections were used and suffered only minor damage during the tests.

### INTRODUCTION

Many researchers have considered the design of steel portal frames for static loading by either elastic or plastic design methods, but few have studied their behaviour under high intensity seismic loading. Due to their low mass and height, the design of the majority of portal frame structures is governed by non-seismic load cases such as gravity, wind and crane loads. In such cases the significance of seismic loads may be assessed by assuming a seismic coefficient appropriate for elastic response. This would give an indication of the level of ductility required if the structure is to survive a major earthquake of the intensity implied by the design code. If this approach shows a significant ductility demand then the use of elastic design methods must be questioned. If the ductility demand is high a fully plastic collapse mechanism can be expected to form and the frame should be detailed accordingly using capacity design procedures.

A review of previous test results by several researchers (Beamish 1984) found that local and lateral inelastic stability problems in steel members become more severe if the load is applied cyclically rather than monotonically. Popov (1973) found although local buckling occurred in members satisfying plastic design flange slenderness limits no immediate drop in load capacity was observed. He suggested that more conservative flange and web slenderness limits may be needed for cyclic loading and that the compression flanges should always be braced to prevent lateral-torsional buckling. Whittaker and Walpole (1982) and Matsui and Yoshizumi (1980) found that web buckling following the flange buckling lead to rapid decreases in load capacity. Vann et al. (1973) concluded that beams with flange, web and minor axis slenderness ratios close to the plastic design limits may exhibit unstable hysteresis loops but

deterioration would be severe only when local flange buckling combined with either web buckling or lateral-torsional buckling.

For economy, a typical portal frame will often have the lightest members in a particular size range so it is important to consider the effect of cyclic loading in view of the previous test results. In this paper the results of three tests on full-size portal frame knee joints are presented. The aim of these tests was to evaluate the performance of three different joint configurations and to investigate potential stability problems.

Morris (1981) has reviewed the design of portal frames by both elastic and plastic methods but the only joint type considered was a bolted, haunched joint detailed for a closing joint moment. The most common joint detail used in New Zealand is a fully welded connection with a diagonal stiffener. This detail, along with a bolted end plate joint and a haunched joint with a bolted end plate were tested. The bolted end plate details are similar to those tested by Johnstone and Walpole (1981), Whittaker and Walpole (1982) and Plugge and Walpole (1983).

### TEST SPECIMENS

The 'model' structure used to select the dimensions of the test specimens was a single bay portal frame with a span of 25-30 m, an eaves height of 6 m, a roof pitch of 1 in 5 and partially fixed bases. The chosen leg length of the test specimens (4 m) approximates the points of inflection for gravity loading. The member sizes were selected using a plastic design with the requirement that the sections should be the lightest in their size range so that any problems with local buckling could be investigated. Capacity design procedures were used when designing the bolted connections.

Specimen one (Figs. 1,4) is a typical fully welded knee joint with a diagonal stiffener. Although the lateral restraint

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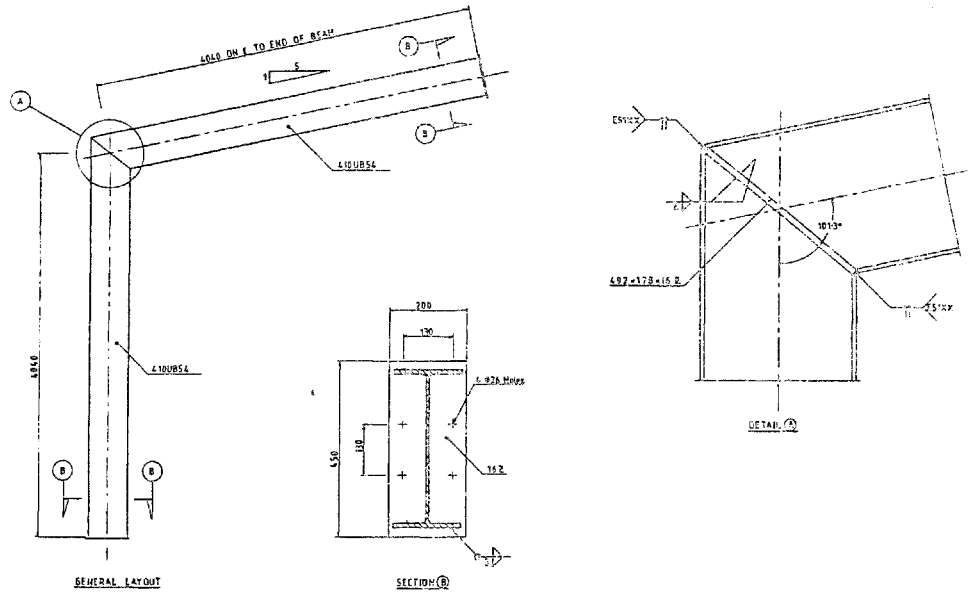


FIG. 1 - SPECIMEN ONE DETAILS

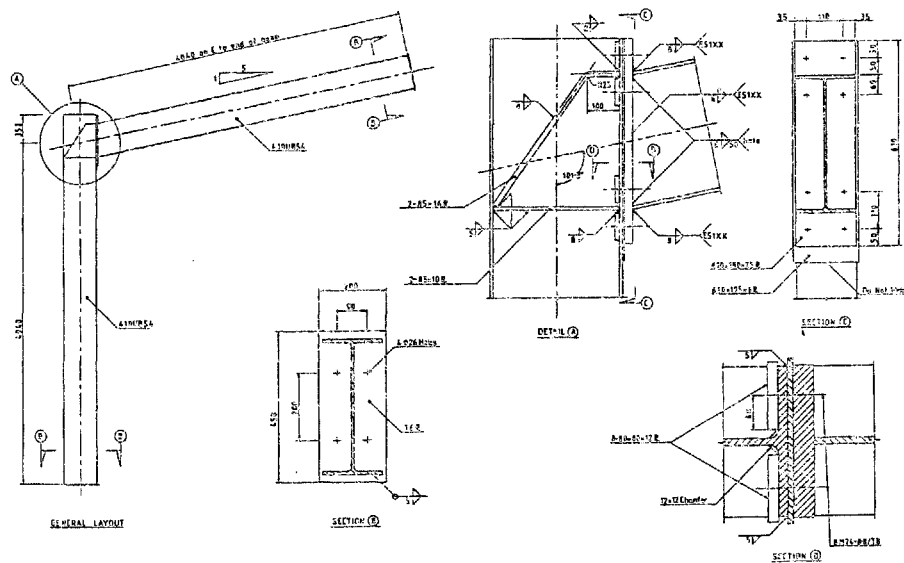


FIG. 2 - SPECIMEN TWO DETAILS

spacing satisfied the rules for plastic design in AS 1250-1981, the expected failure mode was lateral-torsional buckling.

Specimen two (Figs. 2,5) used a bolted end-plate connection with a diagonal stiffener configuration developed by Morris (1981). A thin plate was welded to the outside of the column flange to replace

the area lost at the bolt holes and to stiffen the flange against deformation caused by the bolt tension forces. The 12 mm backing plates provide extra column flange strength - Zoetemeijer (1981) has suggested that a 50% increase in strength can be obtained using plates slightly thicker than the column flange. The beam plastic moment was increased by 25% for the connect-

ion design. The end plate thickness is between the values obtained from the Surtees and Mann (1970) and the Mann and Morris (1979) formulas. The bolts were sized by assuming that the flange force is shared equally by the four bolts and that the prying forces are allowed for by using the proof load rather than the ultimate load. The column flange strength was calculated using a yield-line mechanism suggested by Packer and Morris (1977).

Specimen three (Figs. 3,6) was a haunched joint where the haunch was formed by a triangular off-cut from the beam section. The bolted connection was designed using the same method as for specimen two

but was slightly under-designed (allowing for a 25% over-strength beam). The end plate size was obtained from the Surtees and Mann equation. The lateral restraints were positioned around the possible plastic hinge locations to prevent lateral-torsional buckling. Restraint No. 5 (Fig. 6) was moved during the test to the indicated position when lateral deformations were observed in the vicinity of the top flange connection.

TEST PROCEDURE

The layouts of the test rig and the lateral-restraint positions for each specimen are shown in Figs. 4-6. The specimens

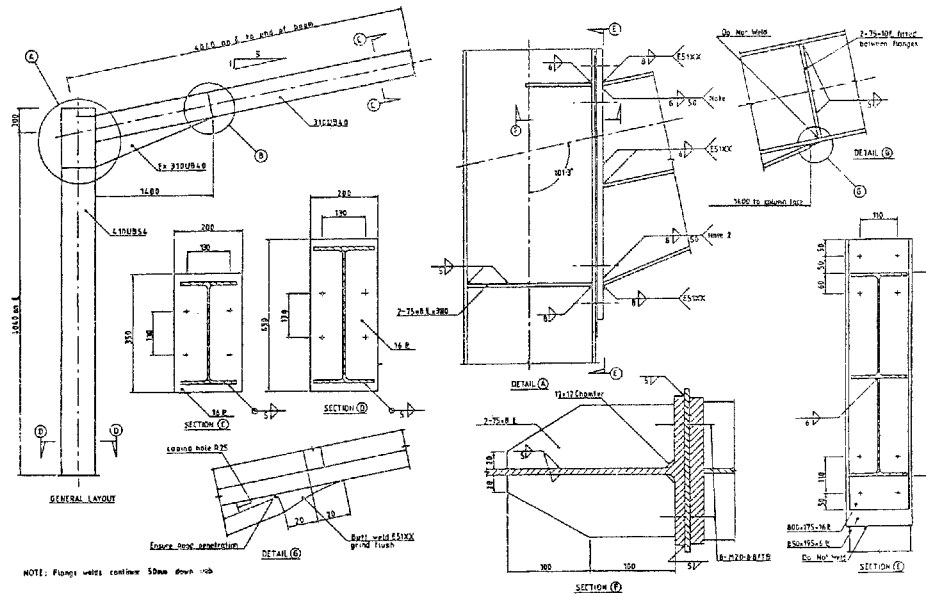


FIG. 3 - SPECIMEN THREE DETAILS

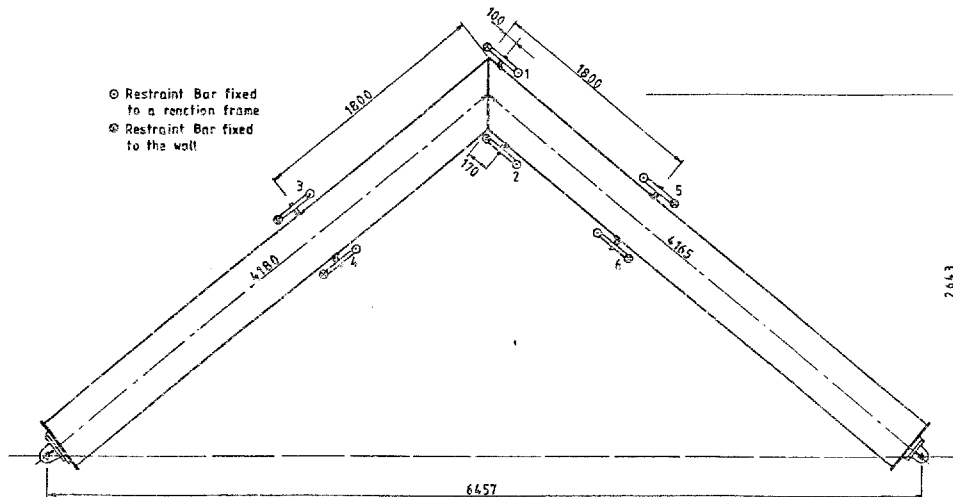


FIG. 4 - SPECIMEN ONE TEST LAYOUT

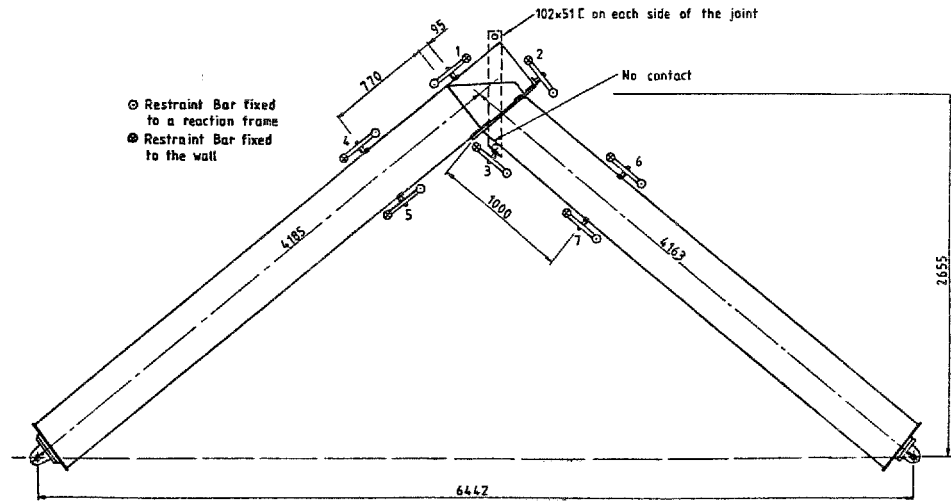


FIG. 5 - SPECIMEN TWO TEST LAYOUT

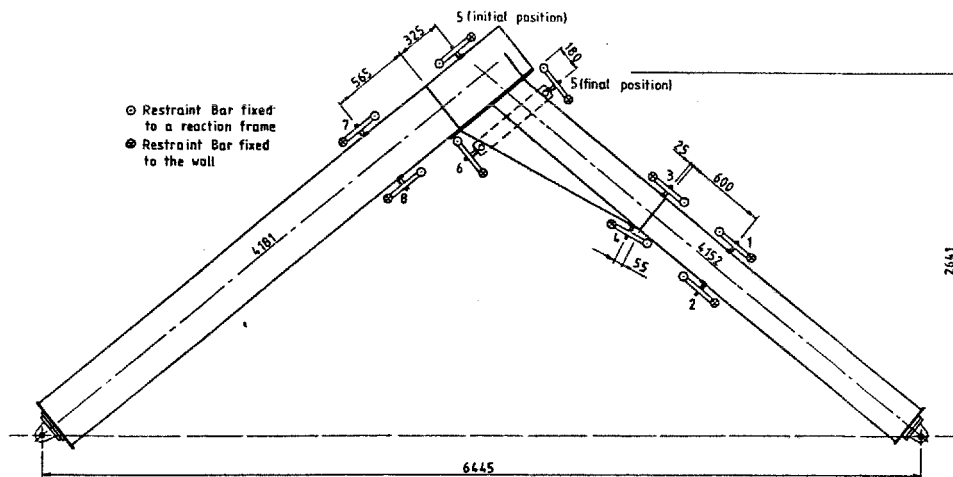


FIG. 6 - SPECIMEN THREE TEST LAYOUT

were set up vertically between a fixed hinge at the end of the column and a hydraulic jack at the beam end. The lateral restraints are a straight line mechanism which allows large deflections in the plane of the specimen with negligible lateral movement. The restraint pivots were attached to the member flanges using clamps or cleats welded to the connection where convenient. The clamps were found later to be unsuitable for use in plastic hinge regions. The restraint bars were instrumented and calibrated to enable the restraining forces to be measured.

Linear strain gauges were attached at various positions along the beams and joint components and rosette strain gauges attached to the web panel areas to measure shear strains. Extensometers were used to measure joint deformations and plastic hinge

rotations. All instrument readings were recorded at frequent intervals on a data-logger system.

A method developed by Surtees and Ibrahim (1980), where the average strain in rosette gauges fitted to bolt heads was calibrated against the force, was used to measure the bolt forces in specimens 2 and 3. When calibrating the bolts it was found that a reliable estimate of the maximum force to which a bolt has been subjected was given by the onset of non-linearity in the load versus strain plot for a subsequent load cycle. After the tests were completed the bolts were re-calibrated to determine the maximum loads reached and their ultimate strength. When tightening the bolts the load was monitored to determine when the proof load was reached. In all cases the load was reached with less than one

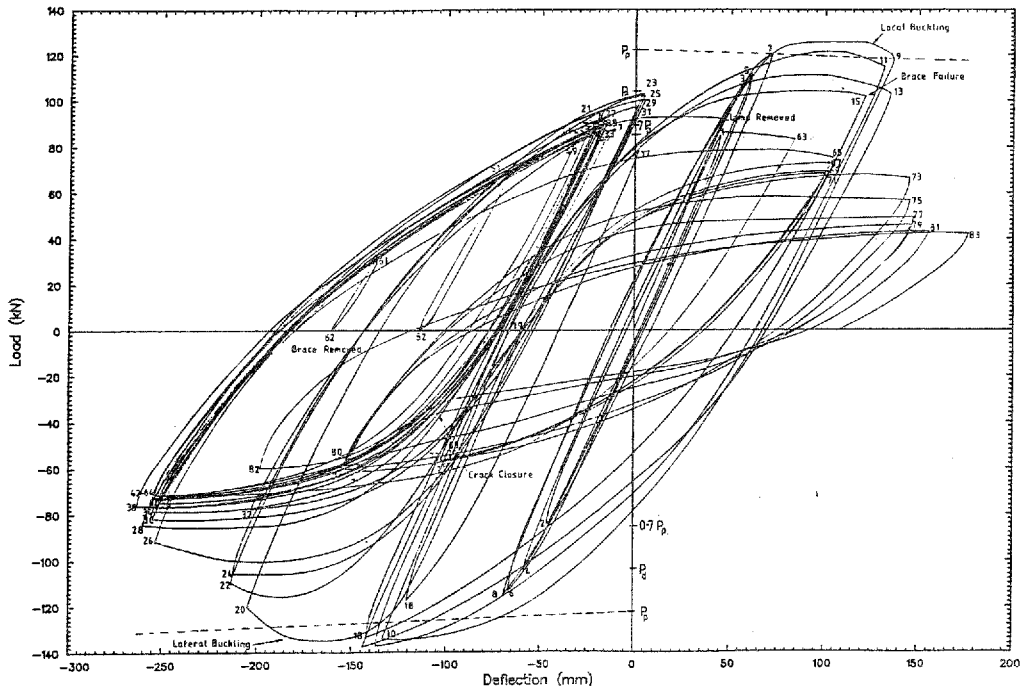


FIG. 7 - JACK LOAD VERSUS DEFLECTION, SPECIMEN ONE

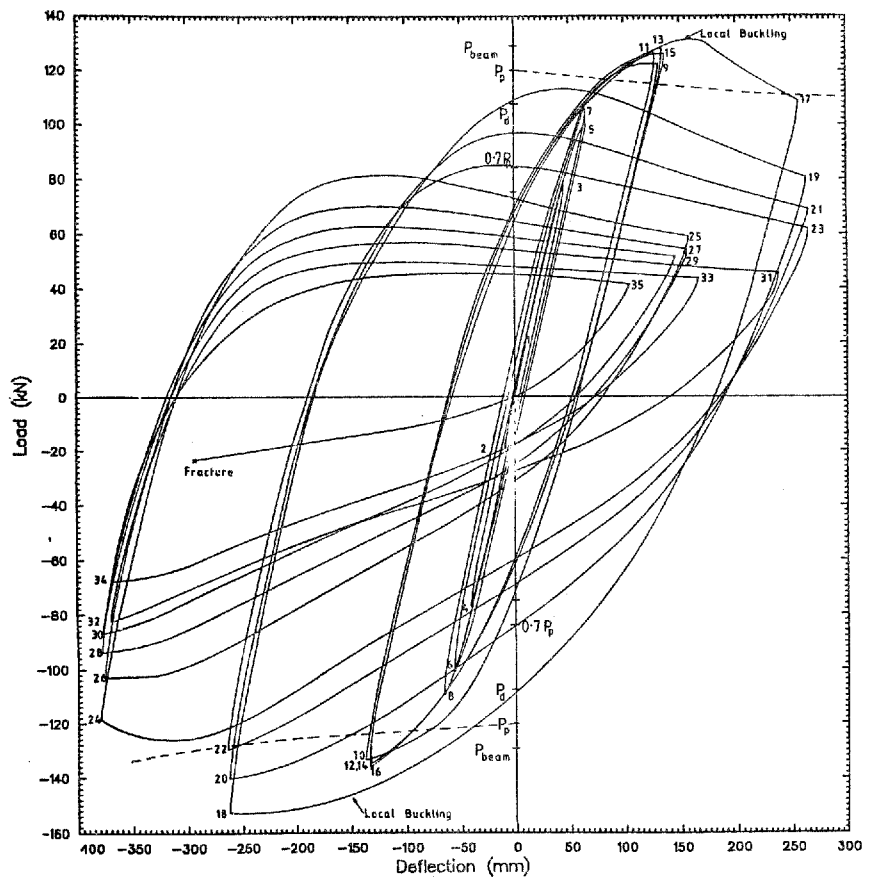


FIG. 8 - JACK LOAD VERSUS DEFLECTION, SPECIMEN TWO

half turn from the 'snug tight' condition if this was assumed to be the load required to bring the plies into contact.

A typical load sequence comprised two or three cycles at a low load level; one cycle at 1/3, 2/3 and then the full predicted yield load; two cycles at ductility factor 1.0 and then further cycles at gradually increasing ductility factor until failure. The deflection at the predicted plastic moment of the weakest member was calculated using the measured stiffness of the specimen. In later cycles the closing displacements in tests one and two were limited to avoid serious damage to the lateral restraint components.

RESULTS

Plots of jack load versus deflection for the three tests are shown in Figs. 7-9 and some of the numerical results are summarised in Table 1. The numbers at the end of each curve segment indicate the excursion number and the dashed lines indicate the calculated collapse load for a rigid-plastic mechanism. The loads at the design

plastic moment and 70% of the calculated beam plastic moment are also shown. The commentary to NZS 4203 : 1976 suggests the use of a 30% reduction in the member strength as the failure criterion.

For comparison with the code limits on flange and web slenderness ratios (AS 1250-1981) the values using the measured yield stresses are indicated in Table 1. The flange ratios are equal to or exceed the allowable value (136). The web ratios all exceed the maximum value allowed by NZS 3404 : 1977 for plastic hinge regions (800) when the measured web yield stress is used. The minor axis slenderness ratios for either the top or the bottom flange in compression are compared to the lower limit given in AS 1250.

The maximum ductility factors given in Table 1 are calculated using the elastic deflection at the beam plastic moment. The maximum member ductility factors are calculated from the measured plastic hinge rotations but in some cases the members could be considered to have "failed" before reaching that ductility factor.

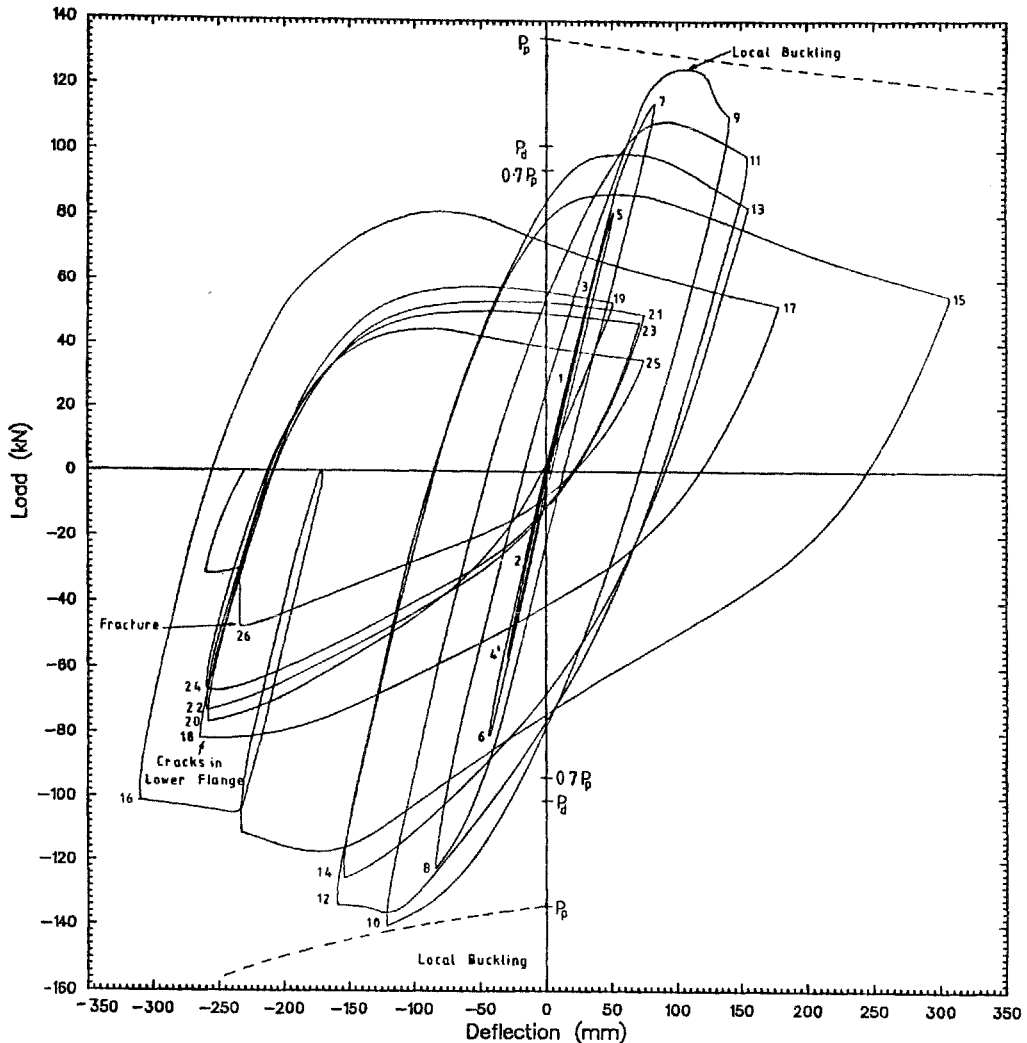


FIG. 9 - JACK LOAD VERSUS DEFLECTION, SPECIMEN THREE

Test Specimen Number	1	2	3
Beam section	410UB54	410UB54	410UB54
Column Section	410UB54	410UB54	310UB40
Beam flange yield stress (MPa)	301	293	303
Beam web yield stress (MPa)	328	351	406
Column flange yield stress (MPa)	301	274	309
Column web yield stress (MPa)	328	328	377
Beam plastic moment: design, actual (kNm)	273,322	273,323	162,216
Column plastic moment: design, actual (kNm)	273,322	273,300	273,350
Jack load for beam plastic moment (kN)	122	129	135
Jack defln. at beam plastic moment (mm)	67.4	66.9	75.2
Beam flange slenderness, $b_1/T$ , $b_1\sqrt{f_y}/T$	7.92,137	7.97,136	7.74,140
Beam web slenderness, $d_1/t$ , $d_1\sqrt{f_y}/t$	50.0,906	49.4,926	49.5,997
Max. permissible axial force (kN)	281	261	127
Beam minor axis slenderness ratios $L/r_y$			
- lower flange	39	25	16
- upper flange	47	23	16
Lower limit for slenderness: $640/\sqrt{f_y}$	37	37	37
Max. +ve deflection (mm)	137	265	307
Max. -ve deflection (mm)	-262	-380	-310
Max. +ve ductility factor	2.0	4.0	4.1
Max. -ve ductility factor	-3.9	-5.7	-4.1
Max. +ve member ductility factor, beam	3.2	6.0	12.3
Max. -ve member ductility factor, beam	-3.9	-8.5	-14.0
Max. +ve member ductility factor, column	1.4	1.9	-
Max. -ve member ductility factor, column	-2.4	-3.5	-
Max. +ve jack load (kN)	126	132	126
Max. -ve jack load (kN)	-138	-153	-140
Max. +ve bending moment in beam (kNm)	337	343	207
Max. -ve bending moment in beam (kNm)	-350	-355	-212
Max. lateral restraint force (kN)	26.5	32	29
Min. ult. bolt force, max bolt force (kN)	-	282,330	196,200

TABLE 1 - SUMMARY OF TEST RESULTS

The local buckling of the lower beam flange in specimen one was initiated by the twisting moment applied to the flange by the lateral restraint clamp during the first cycle to DF = 2. At this point the force in the restraint was 8 kN (about 1.4% of the flange force). In later cycles the local buckling became so severe that a bearing in the restraint assembly failed. In subsequent cycles the displacement in the closing direction was restricted to avoid further damage. The flange buckling was followed by large distortions in the web (Fig. 11) and a rapid increase in the maximum lateral restraint force. Lateral-torsional buckling of the upper flange began in the first cycle to

DF = -3 accompanied by local buckling of the upper flange leading to a large reduction in load capacity.

Specimen two, which was braced to prevent lateral-torsional buckling, showed the best performance of all three specimens. Local buckling did not occur until the first cycle of DF = +4 when buckling started in both beam flanges accompanied by web buckling. After 4 cycles at this displacement the moment capacity for the closing direction had reduced to less than 70% of the plastic moment. The large local buckling deformations resulted in high forces in the lateral restraints - the peak force was 32 kN or

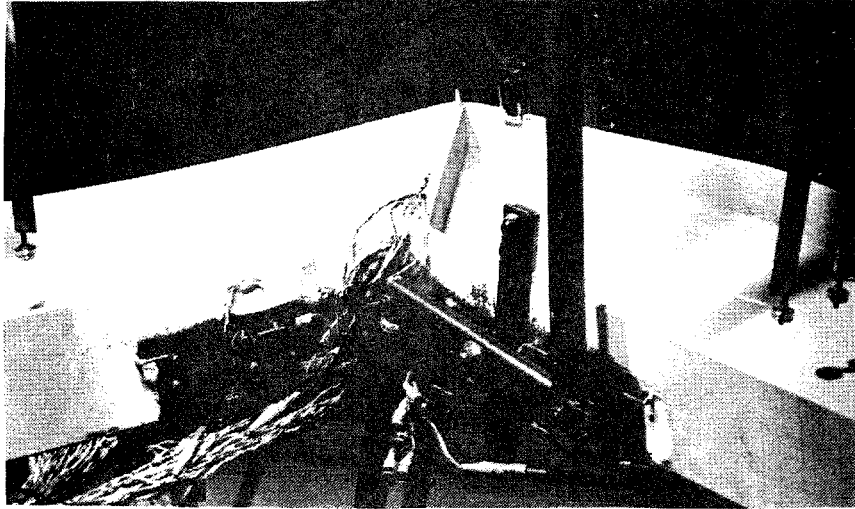


FIG. 10 - LATERAL-TORSIONAL BUCKLING IN SPECIMEN ONE

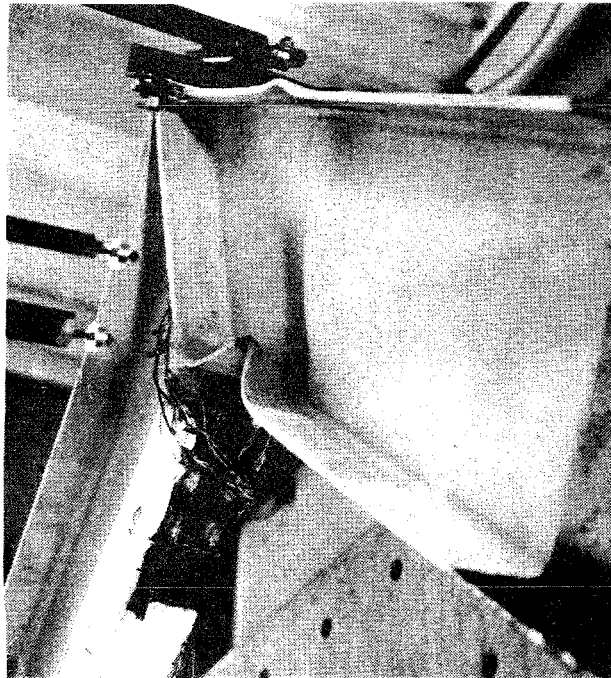


FIG. 11 - LOCAL BUCKLING IN SPECIMEN ONE SHOWN AT TEST COMPLETION

about 5.8% of the flange yield force which is more than twice the design ultimate load specified in AS 1250 (2.5%).

In specimen three almost all of the inelastic deformation was concentrated in the beam at the end of the haunch so that large plastic hinge rotations were required at moderate ductility factors. The member ductility factor at an overall ductility factor of 2.0 was estimated to be 6.1. Three cycles at this displacement level were completed before the moment capacity

fell below 70% of the plastic moment. The calculated plastic moment may not be accurate since the yield stress measured at the centre of the web was quite high (406 MPa) and may not have been uniform over the web depth as assumed. The flange and web local buckling was particularly severe in this test and was reflected in the high lateral restraint forces (29.4 kN or 6.0% of the flange yield force).

The performance of all connection components in the three tests was adequate



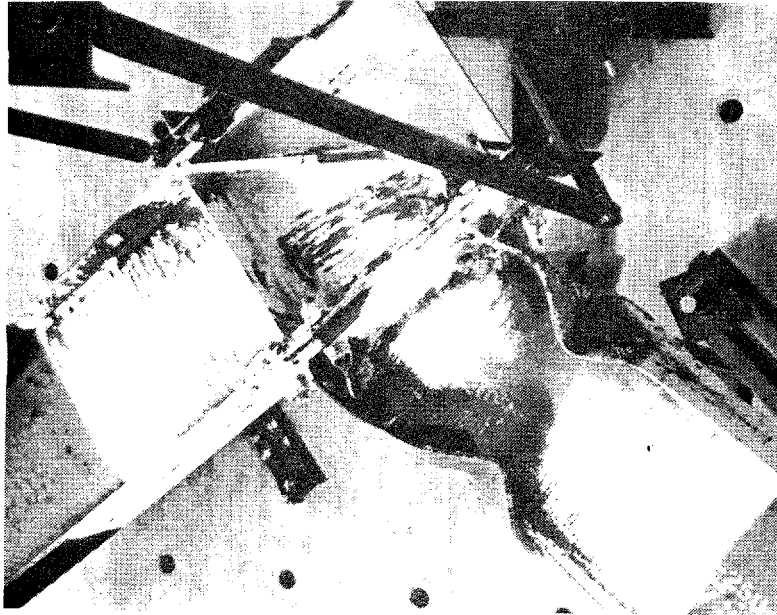


FIG. 12 - SPECIMEN TWO AFTER COMPLETION OF TEST

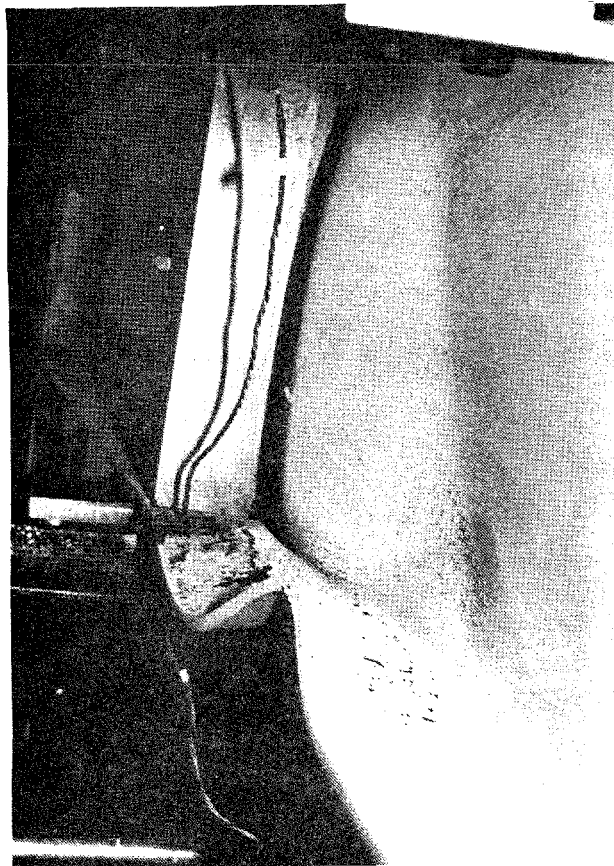


FIG. 13 - LOCAL BUCKLING AT END OF HAUNCH IN SPECIMEN THREE

but it should be noted that the number of cycles at maximum load was limited by buckling in the beams. The maximum separation between end plate and column flange in specimen three was 5 mm which should be considered to be at the tolerable limit for a portal frame structure. In structures where deflection control is more important, thicker end-plates should be used. In specimen two the forces in the bottom row of bolts at the lower flange connection exceeded the specified ultimate loads. Subsequent testing showed that the minimum ultimate strength of the bolts was 347 kN. The maximum prying forces in the bolts were estimated to be 28% of the force calculated by sharing the flange force equally between the 4 bolts (with %5 error margin). The method used to stiffen the column flanges against the bolt tension forces proved to be highly effective.

In all three tests the final failures were by fracture of the beam flanges at local buckles due to low cycle fatigue several cycles after cracks first appeared.

Out of the three specimens tested, only specimen two could be considered to have provided 'adequate ductility' which the commentary to NZS 4203 : 1976 suggests is 8 load reversals at a ductility factor of  $\pm 4$  without the load capacity of any member being reduced by more than 30%. Specimen three would have had 'adequate ductility' if the beam was considered as an isolated member.

#### CONCLUSIONS

Under cyclic flexural loading local flange buckling can occur in members having flange slenderness ratios close to plastic design code limits at relatively low ductility factors. If web buckling also occurs then the local buckling is particularly severe and the load capacity of the member may be substantially reduced. The local buckling is more severe at the inside flange of a portal frame knee joint due to the extra compressive axial force.

Lateral-torsional buckling may also occur after a number of cycles in members where the lateral restraint spacing satisfies the code rules. The distortions and twisting of the cross section caused by local buckling will probably result in earlier initiation of lateral-torsional buckling. The high lateral restraint forces observed in the tests on closely braced specimens where severe local buckling occurred confirms the strong tendency for lateral-torsional buckling to occur in such members.

Increased ductility could be achieved by using stockier members in plastic hinge zones or by reducing the lateral restraint spacing and designing the restraints to withstand the higher forces. The test results suggest that lateral restraints may be required to withstand up to 5% of the flange squash load if severe local buckling occurs. This figure may be compared to the 4% of the flange force recommended by Morris and Randall (1979) for plastic design. If stockier members are used the local buckling would be less severe and the lateral rest-

raint forces would be less but the possibility of premature lateral-torsional buckling remains. Tests by Plugge and Walpole (1983) show that high levels of ductility can be achieved using members with flange slenderness ratios less than those for the tests reported in this paper.

The local buckling which occurred in specimen one show that lateral restraint cleats should not be located in plastic hinge regions unless the flange is suitably stiffened or the cleat is restrained against torsion.

High material yield stresses may have been partly responsible for the earlier than anticipated local flange and web buckling since the critical stresses could have been reached at lower deformation levels. However in a real structure subjected to earthquake loading, overstrength members will, in general, lead to decreased ductility demand so the significance of the higher yield stresses may be reduced.

Portal frames with haunched beams should be designed so that the plastic hinge forms at the top of the column to avoid the high ductility demand in the beam at the end of the haunch. It may be observed that local buckling did not occur in the column of specimen two which was in theory weaker than the beam. It is possible that the end plate and the flange reinforcement plate prevented local buckling.

#### ACKNOWLEDGMENTS

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#### NOTATION AND TERMINOLOGY

- B = flange width                      T = flange thickness
- $b_1$  = flange outstand  $(B-t)/2$     t = web thickness
- D = overall member depth
- $d_1$  = clear web depth between flanges  $D-2T$
- $f_y$  = yield stress
- L = effective length for lateral buckling
- $M_p$  = plastic moment of resistance
- $P_d$  = value of  $P_p$  calculated using specified member properties
- $P_p$  = jack load when plastic moment in weakest member is reached
- $r_y$  = radius of gyration about minor axis
- Flange slenderness: defined by the ratio  $b_1/T$
- Web slenderness: defined by the ratio  $d_1/t$
- Minor axis slenderness: defined by the ratio  $L/r_y$
- Ductility Factor (DF): ratio of the maximum jack deflection to the elastic jack deflection when the beam plastic moment is reached
- Member Ductility Factor: ratio of the maximum transverse deflection at the end of the member to the transverse deflection at the member plastic moment