CYCLIC LOAD TESTS OF COMPOSITE BEAM-TO-COLUMN CONNECTIONS

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SUMMARY

Tests are described of two full-scale sub-assemblages consisting of a steel column and composite beams framed with moment resisting connections. The objective of the study was to investigate the effect of fully reversed cyclic loading on the behavior of the connection which had been designed and detailed for conditions corresponding to moderate earthquake loadings only. In both cases considerable ductility was evident, and resistances were developed which equalled or exceeded those measured on similar specimens tested under monotonic loading.

INTRODUCTION

Floor slabs in modern steel framed multi-storied buildings are usually cast on permanent steel sheet forms, and are frequently designed to act compositely with the steel frame. While considerable experimental data concerning the behavior of bare structural steel members under severe cyclic loading has been accumulated, much less information is available dealing with composite steel-concrete members. In (1) and (2) composite beam tests are described in which the bending moments are symmetric about the columns, and in (3) the more realistic case where bending moments are of different magnitude on either side of the columns is considered. This paper describes tests of two sub-assemblages representing an interior column connection with significant moment transfer between beams and columns. They represent joints in a moment resisting frame, with simple connections in the orthogonal direction.

Seismic design requirements are most frequently specified in terms of load levels only, with little, if any, stated requirement for ductility. Since the design for ductility is thus left to the judgement of the designer, there are wide variations in the amount of ductility available in the members and connections of constructed buildings. Furthermore, while design for ductility may frequently be considered in the most active seismic zones, structures in regions with lesser seismic risk are frequently designed without consideration for ductility, even though seismic loads may govern the design of some members. The member and connection details of the test specimens were therefore not designed to

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exhibit high levels of ductile behavior; they were selected on the basis of resistance to calculated load, and details conform to those used in current practice in Canada.

SPECIMEN DESIGN

The two specimens were nominally identical, and the tests differed only in the applied loading. Fig. 1 indicates the two types of load. For Specimen 1, Fig. 1(a), a load cycle consisted of the load W_1 increasing to a prescribed load or deflection magnitude, and then being reduced to zero; subsequently the load W_2 was applied up to the same load or deflection amplitude and then removed. For Specimen 2, Fig. 1(b), the equal and opposite loads W_1 were first applied and then removed, and subsequently the two loads W_2 were applied and removed. The loads W_2 were increased to attain the same load or deflection amplitude reached in the first half of the cycle. Reactions were provided at top and bottom of the column.

The design of the specimens was based on the assumption that loads were obtained from elastic analysis and therefore plastic design requirements for lateral bracing and beam proportion did not necessarily have to be satisfied. In fact, the selected girder sections are plastic design sections, but the unbraced length of the girders (1871 mm) exceeds the critical length which would be required for a plastically designed beam (1482 mm based on (4) and calculated by ignoring the concrete slab). The unbraced length is short enough however that the plastic moments of the steel section could be attained.

Shear in the column panel zone was estimated on the assumption that the out-of-balance moment to be transferred from girders to columns was equal to the negative bending resistance of the composite section (i.e., the plastic moment of the steel section plus the reinforcing bars). The resulting shear required the addition of web doubler plates of total thickness 12 mm. This detail should be adequate in resisting the loads applied to Specimen 1, but the loading on Specimen 2 was expected to cause distress in this region.

EXPERIMENTAL DETAILS

Test Specimen Details

Details of the test specimens are given in Figs. 2 and 3; these were selected to be typical of current practice.

- the W360X39 girders were fully welded to the column flange using full penetration groove welds.
- the W200X31 transverse beams were simply connected to gusset plates using two ASTM A325 bolts.
- 76 mm deep 20 gage steel deck was oriented along the girder; 8-19mm diameter Nelson studs were welded through the deck in each girder span.
- longitudinal reinforcement of 8 - 15M bars (each of area 200 mm^2) was provided at mid-depth of the 65 mm thick cover slab.
- transverse reinforcement was provided near the column face and near the load points.

- horizontal stiffeners were provided on the girder webs since, according to (5), local buckling was a possibility.
- inclined braces embedded in the concrete provided lateral support to the bottom flange of the girder at the loaded sections.

Load Application

Throughout both tests an axial load equal to 10% of the nominal squash load was maintained on the column. Horizontal movement of column ends was prevented, and the ends were free to rotate.

Cyclic loads at the girder ends were applied by manually operated hydraulic jacks. At each end two load points were used to cause negative bending, the loads being applied to a bridging beam seated on the slab by rods passing through sleeves cast in the concrete. Positive bending was applied by forces applied to the bottom flange of the girder. The loading sequence is described separately for each specimen below.

Instrumentation

Electric resistance strain gages were used to obtain the following data:

- strain distributed throughout the girder depth, at sections close to the columns.
- strains in the reinforcing bars at sections coinciding with the column flanges and at selected points farther from the column.
- strains in the top surface of the concrete slab.
- shear strains at one or more locations on the web doubler plates.

Other measurements included deflections at several locations along the girders and rotations of the column panel zones. Loads were monitored using load cells.

BEHAVIOR OF SPECIMEN NO. 1

The loading and deflection response are summarised in Fig. 4 which shows only the results for one side, since the other side exhibited very similar behavior. Initially, maximum load amplitudes were related to the calculated negative yield moment of the steel section plus reinforcing bars (M_y = 225 kN.m). In the first three cycles, the moment at the column face reached approximately 0.4 M_y, and in the next three cycles was equal to M_y. The first transverse crack in the slab occurred at the end of the third cycle, and further transverse cracks developed in the next three cycles. In cycles 7 to 9 maximum deflections reached approximately twice the measured yield moment deflection, δ_y, and this was followed by cycles 10-15 at $3\delta_y$, cycles 16-19 at $4\delta_y$ and a final loading to much higher deflections.

Distress in the steel section was first observed at the peak load in cycle 7. A slight lateral displacement of the compression flange on the north side was visible, and when the south side was loaded in this same cycle, both lateral displacement and local flange buckling were observed.

This was located at the section where the horizontal web stiffener terminated. At this load the calculated negative plastic moment, had just been exceeded at the column face. The two subsequent load cycles with this deflection amplitude sustained slightly lower moments, but in cycle 10 further increase in resistance was evident as the deflection was increased further. At this point, severe lateral displacement was evident, and all subsequent cycles recorded a degeneration of the resistance. Fig. 4 also shows results of a test on a nominally identical specimen in which loads were applied monotonically. Peak loads obtained are very similar in both tests, with the resistance dropping below the monotonic result only after the 10th cycle.

Shear strains in the column panel zone are shown in Fig. 5. The location of these measurements was approximately at the same level as the plastic neutral axis of the girder, and they therefore represent values close to the maximum anywhere in the panel. The broken lines represent the analytical results using (6). The result labelled k = 1.0 assumes 100% effectiveness of the doubler plates and it appears that k = 0.8 corresponding to 80% effectiveness gives a better prediction of panel zone yield.

The crack pattern at the end of the test is shown in Fig. 6. Regular spacing of transverse cracks, and some longitudinal cracking over the steel girder are evident.

In summary, this specimen, which was subjected only to negative bending, attained its corresponding plastic moment, but subsequently exhibited only limited ductility. For the first 10 cycles, the cyclic load envelope was very close to the monotonic test result, and only after easily visible lateral and local buckling did the cyclic loading cause degeneration of resistance.

BEHAVIOR OF SPECIMEN NO. 2

Fourteen cycles of load were applied to this specimen, and the full load vs. deflection response is given for one side in Fig. 7. The calculated yield moment M_y in negative bending, (266 kN.m) was reached in the sixth cycle, the first two reaching 0.5 M_y and the subsequent ones having progressively increasing load amplitudes. In cycles 7 to 9 maximum deflection amplitudes were $2\delta_y$, in cycles 10 to 12, $3\delta_y$, and in the last two cycles they were $4\delta_y$. Fig. 7 illustrates that the cyclic load response was quite stable until the last two cycles, and that the resistance did not fall below that of a nominally identical specimen tested under monotonic loading.

Crushing of concrete at the column flange first occurred in cycle 7 (deflections of $2\delta_y$), and this was followed by cracks extending between the tips of the column flanges. In this and subsequent cycles gaps of 3 to 5 mm opened up between slab and column face under negative bending moment Severe spalling of concrete at the column flange occurred in the 10th cycle.

Much of the deformation in the steel framing was concentrated in the column panel zone. Fig. 8 shows the variation of shear strain in a doubler plate throughout the first 10 cycles. Very high strains, exceeding $10\gamma_y$ are evident. Unlike specimen No. 1 the doubler plates appear to be fully effective when results are compared with the analysis of (6). The panel zone remains stable for strains which are far higher than the maximum recommended in this reference ($4\gamma_y$). Results are again shown for a strain rosette located approximately on the beam neutral axis.

It is recommended in (7) that the positive moment resistance at a column be calculated on the basis of an effective width of slab equal to the column flange width, and with a concrete compression stress of 1.3 f_c' at ultimate. This moment (356 kN.m) was not reached, nor, under negative bending, was the plastic moment reached (361 kN.m), since the resistance of the column panel zone governed the maximum loads carried.

Severe spalling of the concrete around the column occurred, together with considerable longitudinal cracking. Strain measurements on the reinforcing bars showed very inconsistent results which is thought to be due to the very high bond stresses and consequent bond failures due to the rapid variation of moment near the column. Final unloading of the specimen occurred due to lateral buckling of the beam on the north side.

CONCLUSIONS

The resistance of Specimen No. 1 exceeded the negative plastic moment based on the steel section plus reinforcing bars by 13%, and only after this moment had been reached did the expected lateral buckling cause deterioration of resistance in succeeding load cycles. Specimen No. 2 demonstrated the stability and high energy absorbing capacity of the column panel zone, and as in the case of Specimen No. 1, the resistance closely approximated that measured under monotonic loading. For Specimen No. 2, the positive moment resistance at the column, calculated according to (7), was not reached, although considerable crushing of concrete occurred at the column face. This is partly due to the yielding of the column panel zone which limited the moments carried by the girders. The similarity between the results for cyclic and montonic loads is also evident in the tests described in (3). The limited ductility of Specimen No. 1 and the higher sustained ductility of No. 2 are related to the observed failure modes in the same way as in the case of non-composite beam-to-column connections.

ACKNOWLEDGEMENTS

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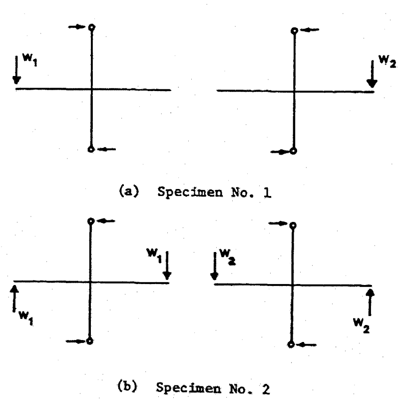

FIG. 1 LOADING OF TEST SPECIMENS

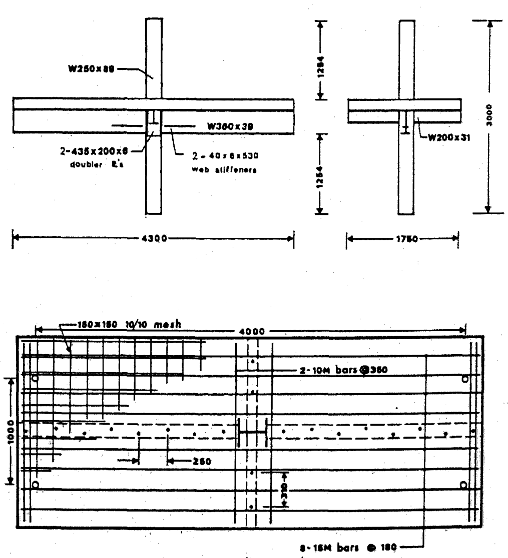

FIG. 2 SPECIMEN DETAILS

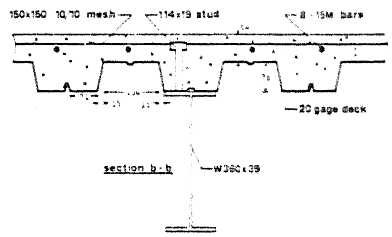

FIG. 3 GIRDER AND SLAB DETAILS

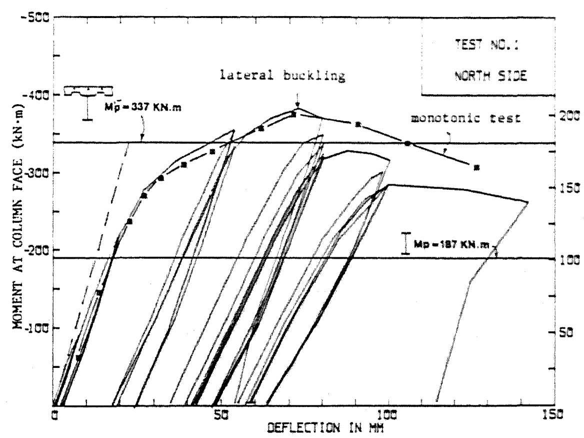

FIG. 4 SPECIMEN NO. 1 - MOMENT VS. DEFLECTION

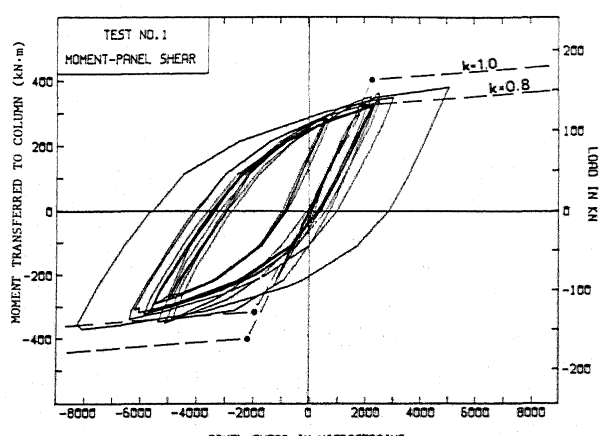

FIG. 5 SPECIMEN NO. 1 - PANEL ZONE SHEARING STRAINS

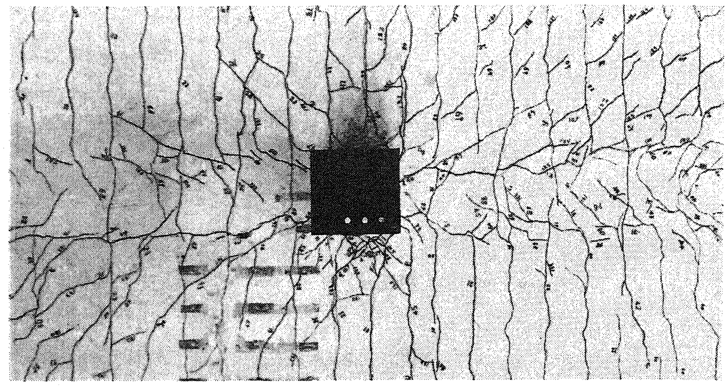

FIG. 6 SPECIMEN NO. 1 - CRACK PATTERN AFTER 20 CYCLES

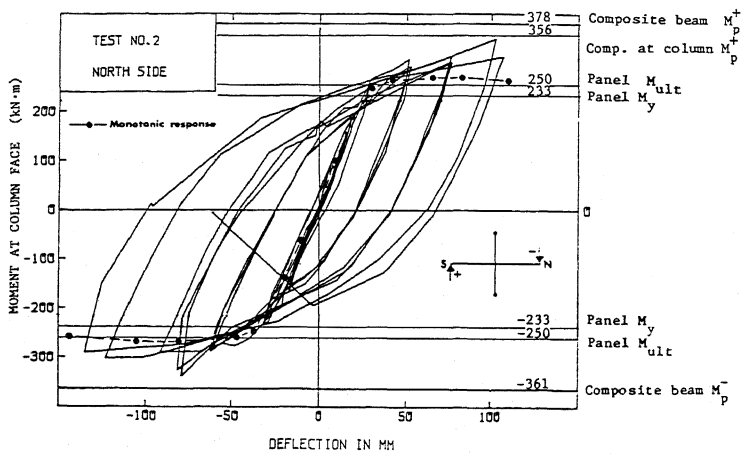

FIG. 7 SPECIMEN NO. 2 - MOMENT VS. DEFLECTION

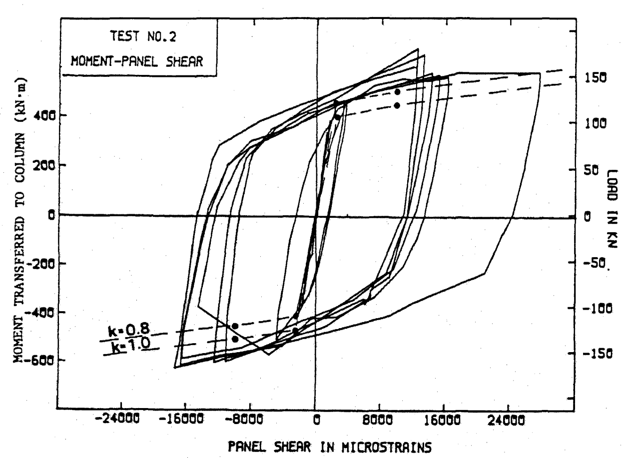

FIG. 8 SPECIMEN NO. 2 - PANEL ZONE SHEARING STRAINS