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EFFECT OF FLOOR SLABS ON THE PERFORMANCE OF SMR CONNECTIONS

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ABSTRACT

Numerous welded steel moment-resisting connections failed by brittle fracture during the Northridge, California earthquake of January 17, 1994. These failures are unique in the history of modern steel construction, and they have led to extensive testing and a careful reexamination of the current provisions for the design of such connections. The results of tests of two full-scale specimens of a typical interior moment-resisting connection, one a bare steel specimen and one with a composite floor slab, are described in this paper. The preliminary results indicate that the strains near the bottom flange of the specimen with a slab are significantly larger than those near the top flange. These results help explain the predominance of bottom flange failures, and they indicate that changes may be required in design codes which currently ignore the effects of unintended composite floor action.

KEYWORDS

Steel connections; welded connections; brittle fractures; backup bars; composite construction; composite floors; floor slabs; slab reinforcement.

INTRODUCTION

The Northridge, California earthquake of January 17, 1994 caused a large number of steel moment-resisting (SMR) connections in steel frame structures to fail. These connections underwent very few large cycles of loading (perhaps as few as two) early in the load history before they failed. In the event that the earthquake had contained one or more large cycles later in the load history, it is possible that these structures could have suffered large permanent offsets or collapsed. The type of damage observed in many of the surveyed structures is unique in this country and has been shown only in a few test series in the laboratory (Uang and Bertero, 1986; Tsai and Popov, 1988).

The damaged connections typically consisted of deep I-girders (W27 to W36) framing into the flanges of heavy wide-flange columns (W14x145 and heavier). In this type of structural system, especially popular in Southern California, very few moment-resisting frames are used to resist lateral loads. The heavy spandrels are needed to control drift, and the requirement that the summation of moment capacities at the joints exceeds that of the framing girders results in the need for heavy column sections. The girder flanges are connected to the column flanges using full penetration welds and details that were thought to provide excellent ductility and toughness (AISC, 1992). Single plate shear tabs are usually bolted to the girder web and welded to the column flange, with some welding to the girder web required if the girder flanges cannot transfer more than 70% of the plastic moment capacity of the cross-section. Interestingly, although many steel frames suffered damage to a large number of their moment-resisting connections, the structures themselves showed little or no external damage or evidence of having been subjected to large drifts. In fact, it was not until several months after the event, when extensive damage surveys were conducted, that the magnitude of the problem was established (Youssef et al., 1995).

The failures observed occurred in frames with a great variety of geometrical configurations, and often in frames that were only a few years old, indicating unexpected shortcomings in current design and detailing procedures. Much of the damage that was observed in the frames concentrated on brittle failures of the welds, HAZ, or column flanges and webs, with the failures starting at or near the full penetration welds connecting the bottom girder flange to the column (Fig. 1).

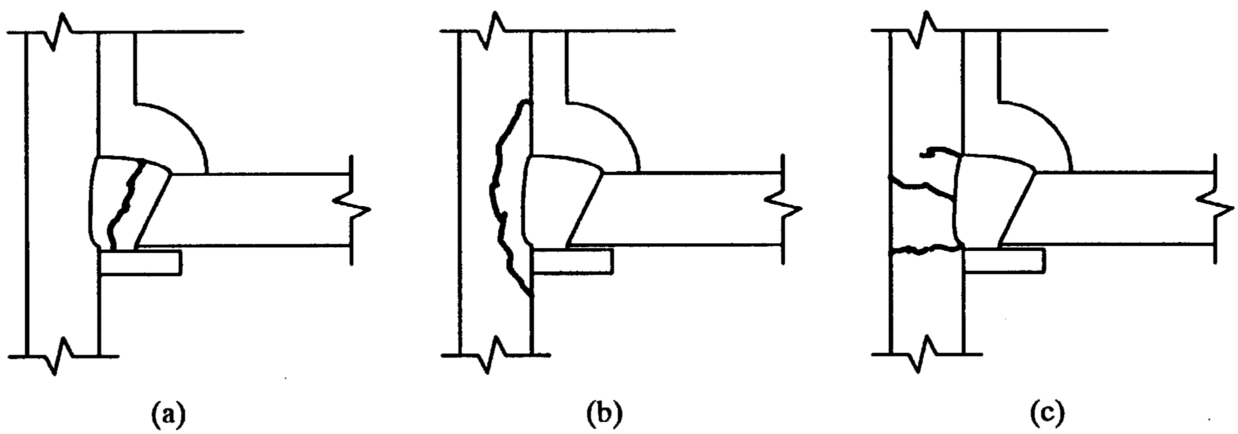

Fig. 1. Typical failures for SMR connections: (a) full or partial crack through weld metal or HAZ; (b) flange tear-out or divot; and (c) partial or full column flange or web crack (after (Youssef et al., 1995)).

While poor weld workmanship, inadequate weld materials, and difficulties in inspection may have played a role in these failures, it is likely that unintended structural performance was a significant contributor. Although the data is still incomplete, reports available at this time have indicated that only a small percentage of failures have occurred in top flange-to-column welds (Youssef et al., 1995). The clear bias shown of the failures occurring consistently near the bottom flanges of the girders indicates that there must be a structural or metallurgical explanation for the failures, or a combination of both.

The research described in this paper intends to determine if the "directionality" of the failures is predominantly a structural phenomena (specifically, the effect of the composite floor slab) and not a metallurgical one (i.e., the effect of the presence of a weld backing bar, or poor weld workmanship due partially to difficult access to the bottom flange weld). The three primary issues addressed in this work are:

1. Steel moment frames which resist lateral load are rarely designed as composite systems. However, the presence of shear studs and slab reinforcement needed for both serviceability (e.g., deflection, creep, and shrinkage control) and ultimate strength (e.g., diaphragm action and drag struts) most likely activates different force-transfer mechanisms at the top and bottom of composite girder-to-steel column connections. The presence of the slab also can result in substantially higher beam strength and stiffness than anticipated in design, resulting in proportionally larger forces in the welds and in the columns.
2. The Northridge earthquake subjected buildings to exceptionally large vertical and horizontal ground motion. Engineers observed that many of connections which failed did so on the lower floors of the structures. Therefore, due to a combination of overturning moment and vertical excitation, it is possible that the axial force in the columns was tensile when the column flanges and panel zones failed.

3. Modern seismic steel design codes permit yielding of the panel zone to dissipate energy during an earthquake. The inherent stability and strain-hardening characteristics of this type of yielding are ideal for seismic applications. Such yielding, however, is often associated with large strains, particularly near the corners of the shear panel. In addition, the distribution of strains in the panel is non-uniform and could require an increase in the ductility demands in the weld and column flange.

The ductility demands in these moment connections are proportional to the amount of force being transmitted and to the deformations required. In the case of the top flange of the girder, if a significant portion of the load is transferred through the slab, the force and deformation required at the top weld will be greatly reduced. The ductility is also affected by the flexibility of the shear panel, which results from the panel zone yielding and dissipating energy. With this substantial overstrength at the top of the girder due to the slab, it is likely that large forces will concentrate in the bottom flange of the girder and in its weld to the column. The influence of the slab on the strain demand at the top and bottom flanges is the primary topic of this paper.

EXPERIMENTAL WORK

To study the influence of floor slabs on the strength, stiffness, and fracture performance of SMR frames, three specimens were tested at the University of Minnesota. The first, Test 1, was a bare steel specimen to be used for baseline comparisons. The other two incorporated a 60 in. wide, 3 in. slab on a 2 in. metal deck. The difference between the composite specimens is in the amount of composite action: Test 2 was designed for fully composite action while Test 3 was designed for 50% composite action. The composite action was governed by the strength of the slab, whose size was constrained by the test setup. Thus the term "fully composite action" as used here does not imply that the steel section would be able to reach full yielding in tension. Mill reports indicated yield strengths of approximately 47 ksi and 57 ksi in the girder and column, respectively. Figures 2 and 3 give a detailed view of the test setup used and the connections tested. Only comparisons between Tests 1 and 2 will be given in this paper. Welds are not shown in Fig. 3, and the deck is drawn larger than scale for clarity.

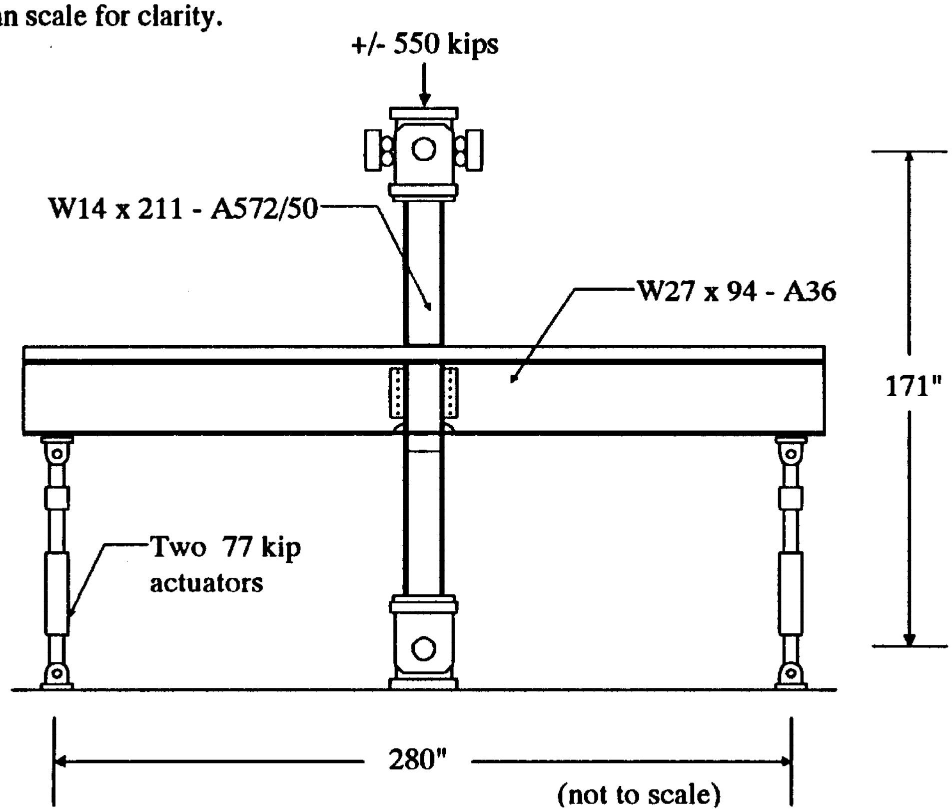

Fig. 2. Overview of specimen and test setup.

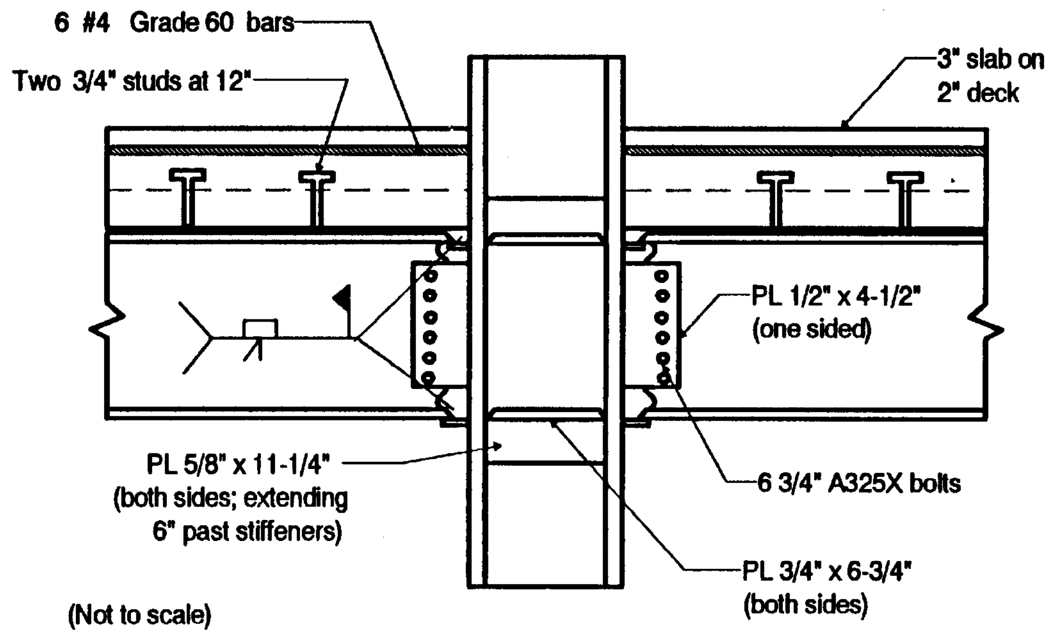

Fig. 3. Details of connections tested (Note: Test 1 did not have a slab).

These tests were the first ones on interior connections carried out after the Northridge earthquake. It should be noted that the W27x94 beams used are at the limit (just over 70% of their capacity comes from the flanges) where welding of the shear tab would be required by codes. The tests have the additional features of incorporating not only a floor slab but also a tensile load in the columns, since initially it was felt that the vertical accelerations may have contributed to the large number of unusual failures.

All the welds were executed as per the applicable AWS standards. Initially, all backing bars were left in place. However, after non-destructive tests indicated some small flaws in the welds, several of the backing bars were removed and the defects repaired. In the case of Test 1, this resulted in the West beam (as it will be referred to) having backing bars at both the top and bottom, while the East beam had a backing bar only at the top. For Test 2 only the top West connection had the backing bar removed.

EXPERIMENTAL RESULTS

The tests were conducted by subjecting the specimens to reversed cycles of load at interstory drift levels of 0.1%, 0.25%, 0.50%, 0.75%, 1.0%, 1.5%, 2.0% and 3.0%. Two cycles were applied at each of these load levels, unless there was damage observed at the second cycle at a particular interstory drift. In these cases, the specimen was subjected to a third cycle to determine if further damage would occur. Thus, three cycles were applied in Test 1 at 1.5% and 2% drift levels, and in Test 2 at 0.75%, 1.0%, and 1.5% drift levels.

Figure 4 shows the beam end load versus beam end displacement curve for the West beam of Test 1. The bottom weld on this beam, which had a backing bar attached to it, fractured early in the test, during the second full cycle of loading at 1.5% drift (in the figure, 1.4 inches of beam displacement corresponds to 1% interstory drift). The failure was similar to that shown in Fig. 1(a), except that it occurred just outside the weld in the HAZ between the weld and the column flange. The specimen gave very little evidence of the impending failure except for slight bowing of the backup bar, which started at a drift of 1% and probably indicated that a crack had begun to grow at the root of the access (cope) hole. Yielding had begun at the 0.75% story drift level, as evidenced by the flaking of the lime coating applied to the beam and in the strain data, but at 1.5% drift, the only substantial yielding that had taken place was in the flanges, within a few inches of the welds. Thus, the pattern of yielding indicated that the beam had not reached anything near the plastic hinge stage. The West beam only achieved a rotation of 0.0068 radians before failing.

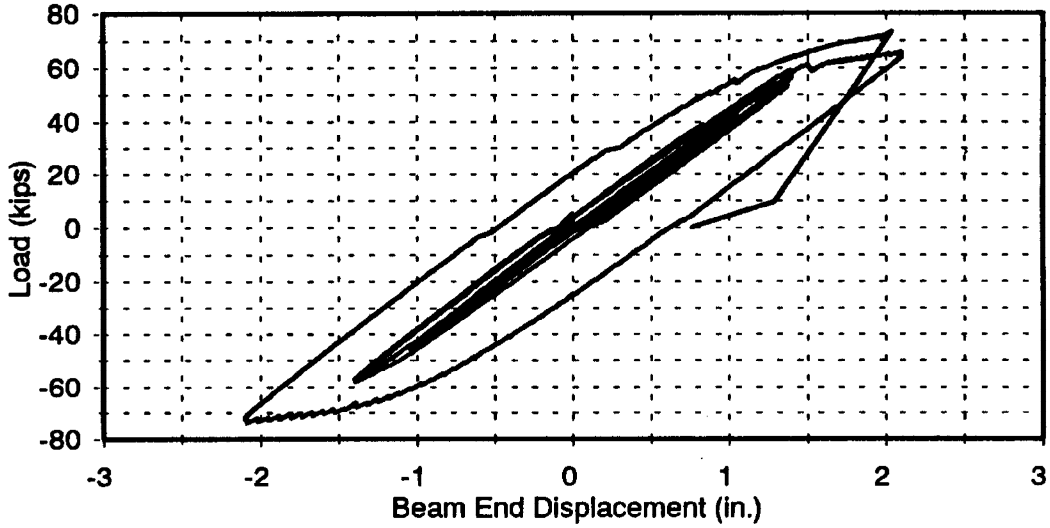

Fig. 4. Load-deflection curve for West Beam (Test 1).

After the failure of the West beam, the actuators on that beam were disconnected and testing was continued on the East beam as a cantilever specimen. Figure 5 shows the behavior of the East beam throughout its displacement history. The connection on the East side had a backing bar on the top but no backing bar on the bottom. As shown by Fig. 5, this connection behaved much better. At the 1.5% and 2.0% drift levels, there was substantial yielding observed in the beams, with some of it propagating into the web and around the bolt holes in the beam web. There was a small buckle that began forming in the bottom connection at the 2% drift level. This buckle grew substantially during the first loading cycle at the 3% drift level and led to the formation of a large crack in the base metal of the girder upon load reversal. The crack initiated in the area of stress concentration on the top side of the bottom flange at the root of the access hole. It propagated predominantly to one edge of the beam flange at the flange-weld intersection. This failure was due primarily to low-cycle fatigue and occurred after the connection had undergone several cycles at rotations of about 0.012 radians, but it only sustained a rotation in excess of 0.025 radians during the first excursion to 3% drift.

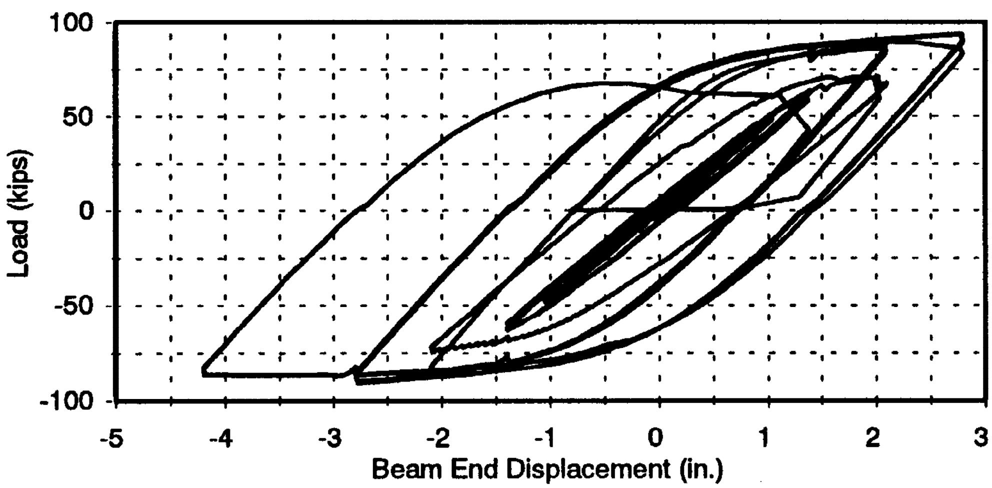

Fig. 5. Load-deflection curve for the East beam (Test 1).

Figure 6 shows the strains measured from five strain gages located at a distance of 13.5 in. (beam depth/2 = D/2) from the welded connection. The gages were located at the extreme fibers, at the quarter depths, and at the centerline of the section. The load stages shown correspond to the 0.5% and 0.75% story drift levels.

On a first inspection, the data indicates that at this cross-section an assumption of plane sections remaining plane, such as is made in design for the determination of the connection forces, is reasonable. The calculated strains from an elastic analysis at these stages would have been about 490 $\mu\varepsilon$ for the 0.5% drift level and 770 $\mu\varepsilon$ for the 0.75% drift level. A closer inspection reveals subtle but important deviations. The bottom flange shows significantly larger strains when loaded in tension (positive strain) than compression, while the top flange shows larger compressive strains than tensile. However, the largest tensile strain in the bottom flange is consistently larger than any top flange strain. Also, the strain gage at the web centerline, which should remain at zero strain, shows a steady migration towards the tension side of the bottom flange, thus indicating a neutral axis shift. Even in the bare steel specimen, the placement of the shear tab much nearer to the top flange may lead to more excessive straining of the bottom flange.

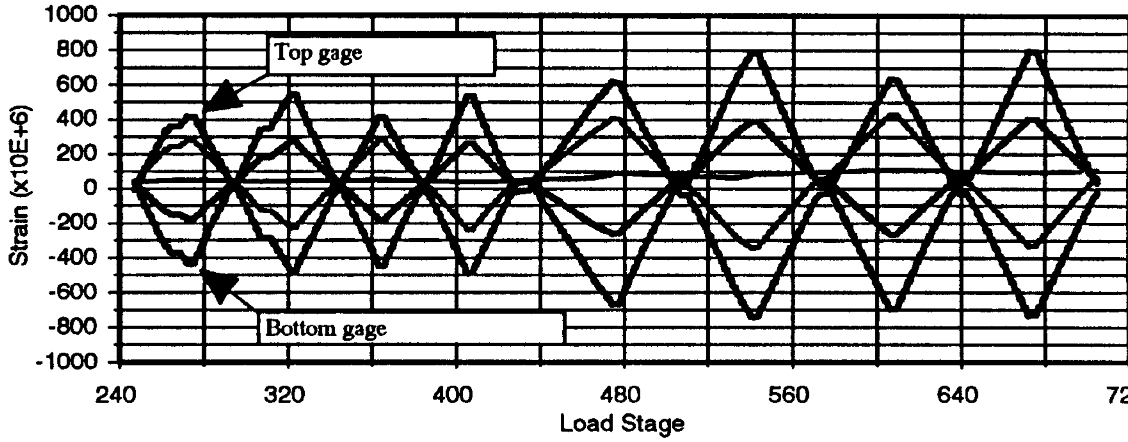

Figure 6. Strains at D/2 for the East Beam (Test 1).

Figure 7 shows a comparison of the strains measured on the bottom flange at D/2 with those measured at a distance of 1.0 in. from the weld utilizing a high elongation strain gage. Linear elastic theory would have predicted that the strains at the weld would have been roughly 11% higher. Figure 7 indicates that they are much higher, on the order of three times larger. This indicates a substantial strain concentration near the weld. Detailed finite element studies (performed at the University of Minnesota as part of this research) utilizing a very fine discretization in this area indicate that stress concentration factors on the order of 2.0 to 2.5 are to be expected in this region, and that larger values (on the order of 3 to 4) can be expected at the weld/column flange interface. The important conclusion from these measurements is that the local ductility demands are very high, since most of the stress concentration occurs within 2 to 3 in. from the weld. From a practical standpoint, these stress concentrations also mean that it is unlikely that the full plastic moment capacity can be reached for beams near the 70% flange moment capacity limit even considering the effect of strain hardening. For the W27x94 beams used in these tests, for a yield strength of 47 ksi, the data indicates that yielding started at a girder tip load of 35 kips rather than the 94 kips calculated from linear elastic theory.

The results from Test 1 discussed above are not surprising qualitatively. They imply, however, that the presence of a floor slab may result in worse performance since under reversed cyclic loads the bottom flange would be expected to see higher strains than the bare steel one even in the elastic region. Test 2, which included the concrete slab, was designed to help detect whether there were significant behavioral differences between the bare steel and composite specimens. The shear studs for Test 2 were detailed to provide as much composite action as possible without resulting in excessive congestion. Thus, two 3/4 in. studs spaced at 12 in. were used, resulting in an increase in moment capacity of roughly 45% over that of the bare W27x94, and a neutral axis due to positive moment approximately 7 in. above the girder's centroidal axis.

Figures 8 and 9 show the beam end load versus displacement curves for Test 2. In Test 2, the overall behavior was not entirely dissimilar to that of Test 1. One side, the East beam (Fig. 8), again failed brittlely at low levels of drift, but in a two-stage fashion. The first crack, which was initially hard to detect from the surface except for the bowing of the backup bar, originated at the access hole near the top of the first loading cycle to 1.5% drift. The initial crack, which propagated only about halfway to the edge of the flange, resulted in a moderate loss of strength (about 20% of total capacity). Since the crack occurred at about 1.4%

drift, the specimen was displaced further (to 1.5% drift) and the load held for a significant period of time (15 minutes) without the crack propagating further. However, the connection failed completely during the second cycle at this same deflection level. The fracture was very similar to that in Test 1 (West beam -- Fig. 1(a)) and resulted in a decrease in capacity of about 75%. Further cycling of this beam (Fig. 8) resulted in a very unsymmetrical behavior, with the connection exhibiting little strength and stiffness when the bottom flange was subjected to tension, but achieving and maintaining its capacity when loaded in compression. The behavior of the West beam, on the other had, was satisfactory (Fig. 9) up to the 2% drift cycles. Further loading at 2% drift resulted in a brittle failure in the HAZ of the column (Fig. 1(a)).

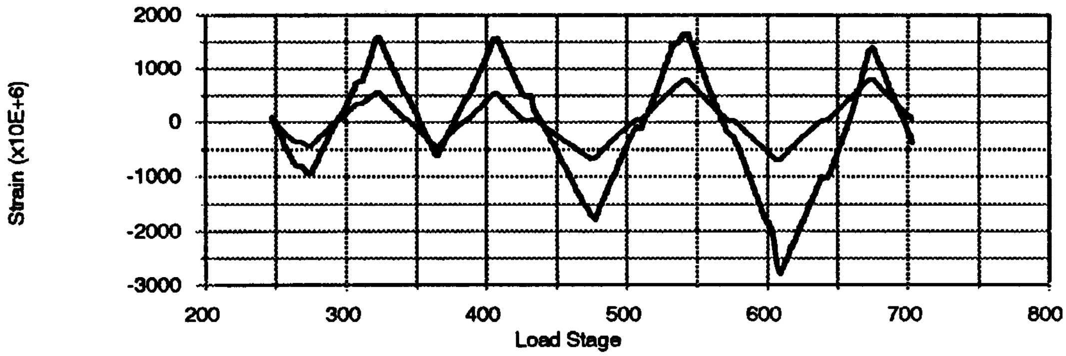

Fig. 7. Comparison of strains at D/2 and at the connection face for the East Beam (Test 1).

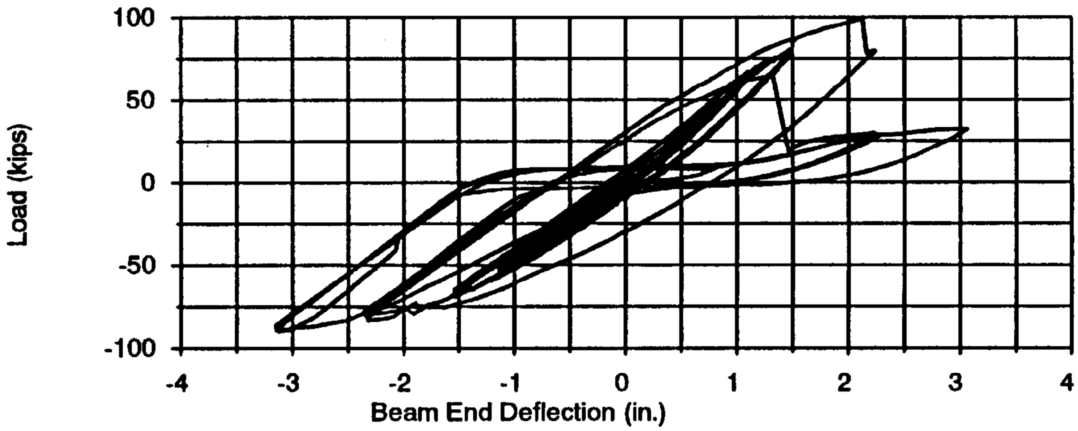

Fig. 8. Load-deflection curve for the East beam (Test 2).

The strain data for Test 2 showed clearly that the neutral axis had migrated several inches upwards when the connections were loaded in positive bending. The neutral axis moved little when loaded in negative moment, indicating that the slab steel was participating little in carrying the loads. A comparison of the strains at the top and bottom flanges of the connection 1 in. from the weld (Fig. 10) shows that there is almost an order of magnitude difference. The data shown is for cycling at 0.75% and 1.0% interstory drift. The strains in the top flange, under the slab, fluctuate by only about 2000 $\mu\varepsilon$ during each cycle and indicate essentially elastic performance. The strains in the bottom flange, conversely, are inelastic and show cyclic yielding early in the load history.

CONCLUSIONS

The results from two tests on an interior SMR beam-to-column connection are presented. The tests include a bare steel specimen and a specimen having a full composite slab. All four of the connections failed at the bottom flange by fracture either at low levels of interstory drift (i.e., 1.5% drift), or prior to a significant number of cycles being executed beyond 2% drift. The preliminary results indicate that the neutral axis of the composite specimen shifts significantly towards the top flange of the girder during positive bending, and that the strains near the bottom flange of the composite specimen are significantly larger than those near the top flange. These results help to explain the predominance of bottom flange failures, and they indicate that changes may be required in design codes, which ignore the effects of unintended composite floor action.

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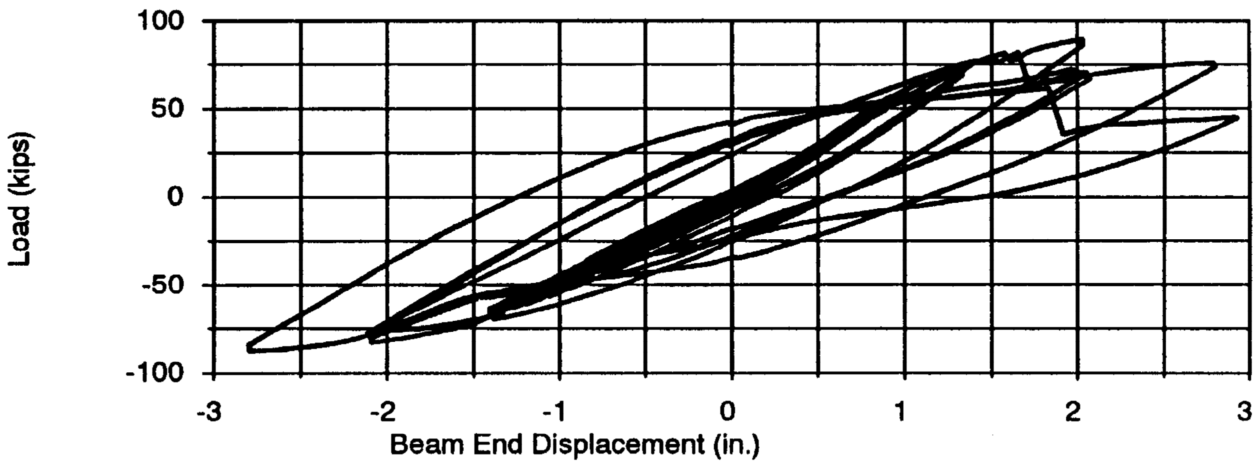

Fig. 9. Load-deflection curve for the West beam (Test 2).

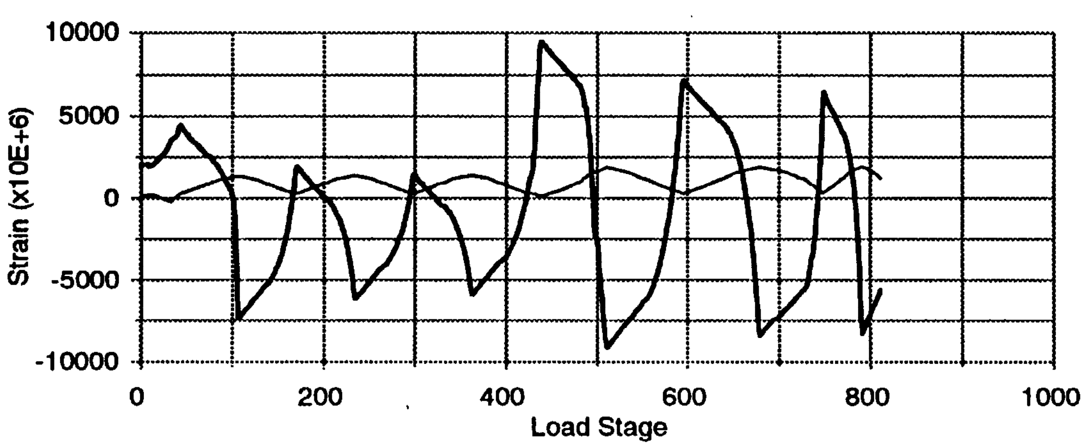

Fig. 10. Comparison of strain at the top and bottom of the flange near the connection (Test 2).