EXPERIMENTAL STUDIES ON THE CYCLIC RESPONSE OF FULL-SCALE STEEL BEAM-COLUMN CONNECTIONS

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SUMMARY

This paper summarises the results of recent experimental work conducted at the University of California at Berkeley (UCB) on the seismic performance of moment-resistant steel connections.

The first project discussed involved testing three typical exterior beam-column connections detailed and fabricated according to pre-Northridge standards. All specimens failed in a brittle manner, reproducing the type of damage observed in the field. They were repaired according to current procedures and tested again. Although the behaviour of the repaired structures was somewhat better than that of the original specimens, the mode of failure was again of a brittle nature.

Several approaches to obtain reliable steel connections are being studied at UCB. One of them involves adding ribs to the beam-column joint. Two specimens with a single rib over the beam web showed stable cyclic behaviour and good plastic rotation capacity. An interesting alternative approach eliminates the beam flange full penetration welds altogether. Portions of heavy W sections are fillet welded to the beam and then bolted to the column. This design is currently being refined and will be tested in the near future.

A relatively simple way to ensure ductile behaviour in beam-column joints consists of weakening the beam by removing material from the flanges (reduced beam sections, or RBSs) in order to induce the formation of a plastic hinge away from the welded connection. Connections with cutouts in the top and bottom flanges and in the bottom flanges only (which could be advantageous for retrofit) were tested and had very good cyclic response. A rigid connection design for a hospital in California which failed qualification tests was modified with RBSs and showed excellent cyclic behaviour. Skillful use of RBSs can therefore improve the behaviour of steel moment connections.

INTRODUCTION

Before the Northridge earthquake, steel moment-resisting frames with welded connections were considered to be reliable systems for seismic resistant construction. This belief, however, was shattered by the numerous fractures in welded beam-column connections caused by this earthquake. The Northridge earthquake effectively invalidated the U.S. codes for moment resisting frame design and construction. The Department of Civil and Environmental Engineering of the University of California at Berkeley (UCB) is an active participant in a nation-wide effort to investigate the reasons for these failures, to establish safe and reliable repair procedures, to recommend retrofit systems, and to develop design codes and construction methodologies. This paper summarises some of the results of the experimental research work at UCB within this research effort in which the authors participated.

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TESTING PROGRAM

Reported tests were conducted at the Structural Testing Laboratory of UC Berkeley. All specimens were full-scale models of beam-column connections. The test set-up was designed to accommodate specimens in a horizontal position. The support system consisted of three reinforced concrete reaction blocks prestressed to the strong floor of the laboratory with high strength rods. The bottom column support restrained displacement in two directions and thus simulated a hinged connection; the top support simulated a roller support.

The load was applied to the cantilever beam end by a hydraulic actuator, through a clevis bolted to the beam end plate. No axial load was applied to the column. To prevent out of plane motion of the beam, a horizontal restraint system was provided near the beam end. Figs. 1 and 2 show the dimensions and basic characteristics of a typical specimen, and a view of a test in progress, respectively.

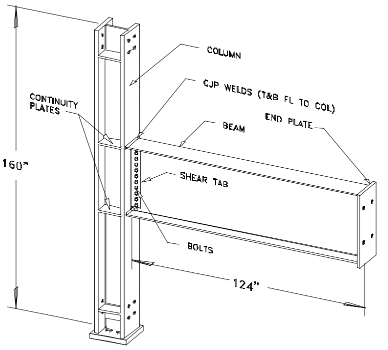

Figure 1: Typical test specimen

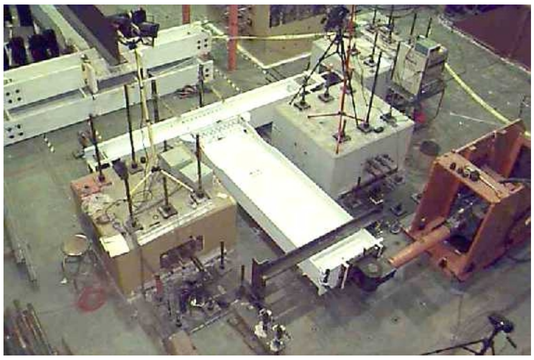

Figure 2: Test set-up

The instrumentation consisted of linear potentiometers to measure beam end deflection, joint rotation, and panel zone shear deformation and strain gages and rosettes, functional within the plastic range, which were glued at critical locations to investigate local response. The specimens were whitewashed prior to testing. Spalling of the steel mill scales due to yielding produced flaking of the whitewash, thus allowing the visualization of yield patterns on the surface of the specimen. Testing was conducted under displacement control, following a standard stepwise cyclic loading protocol based on ATC-24 [ATC, 1992].

PRE-NORTHRIDGE CONNECTIONS

As a result of many failures of steel connections during the 1994 Northridge earthquake, a massive research was launched by FEMA (Federal Emergency Management Agency) under the management and guidance of SAC, a joint venture of the **S**tructural Engineers Association of California, **A**pplied Technology Council, and **C**alifornia Universities for Research in Earthquake Engineering. Extensive literature has been generated by this organisation. Three large-scale external beam-column connections were tested at UCB within the SAC Steel Program [Popov et al., 1996]. The specimens were full-scale models of typical beam-column connections. The details of the connection, the weld specifications, and the erection procedures closely resembled the industry standards in use before the Northridge earthquake. Typically, the beam flanges are directly attached to the column flanges by full penetration welds, which are responsible for transferring the full extent of the beam moment to the column. Shear is assumed to be carried by the shear tab, which is fillet-welded to the column and bolted to the beam. Fig. 3 shows specimen geometry and connection details.

The specimens labelled as PN1, PN2, and PN3 (for Pre-Northridge) were designed with a strong W14x257 Gr.50 steel column and a relatively weaker A36 steel beam. Failure of the beam welds was therefore expected. Testing confirmed the poor response to cyclic loading of the Pre-Northridge design. All connections showed low energy dissipation capacity and failed in a brittle manner, with no warning.

Although the specimens were designed to induce beam failure, in two cases (PN1 and PN2), the failure was in the columns (Fig. 4). This unexpected mode of failure was explained later, when it was determined that, by mistake, the fabricator had made the beams with Gr. 50 steel, instead of the specified A36 steel. This mistake was fortunate, because it was the first time that column failure could be reproduced in a laboratory. It is plausible that this kind of problem can occur in the field, due to the high variability in the material properties of steel available in the US. The third specimen (PN3) failed through the beam full penetration weld at the column face. Such failures were typical in other laboratories.

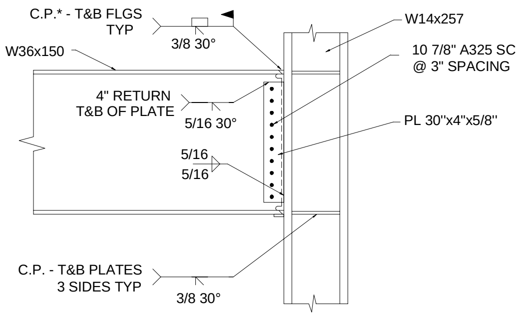

Figure 3: Pre-Northridge connection details

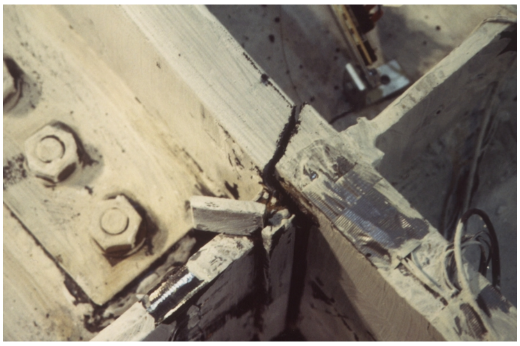

Figure 4: Column failure of specimen PN1

The three specimens were repaired and retested (labeled as RN1, RN2, and RN3). The repair of PN1 and PN3 basically consisted of replacing the material near the fracture with new material. PN2 was retrofitted with an elaborate detail which included a haunch. The repair techniques evaluated were successful in the sense that the repaired specimens were stronger, stiffer, and had better energy dissipation characteristics than the pre-Northridge connections. The most dramatic improvement was obtained for specimen PN2. Fig. 5 shows the retrofitted specimen (RN2) and the moment-plastic rotation response obtained during the tests. None of the specimens was able to develop the minimum acceptable level of plastic rotation of 3%. Furthermore their failure mode was brittle and abrupt, which is undesirable for structures in seismic areas.