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Shear in Reinforced Concrete Beam –Column Joints Without Transverse Reinforcement.

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Overview

Many older buildings have reinforced concrete frames. Their joints often contain no transverse reinforcement because they were designed before the damaging effects of joint shear stress were well understood. Today, engineers are faced with the problem of assessing the capacity of these joints, and possibly retrofitting them to improve their seismic performance. This Digest describes physical tests that were conducted on beam-column sub-assemblages that represent conditions typical of those found in such older buildings. The primary study variables were the joint shear stress demand and the load history. Two different shear stress demands (approx. 10 and 15 f'_c (psi)) and four different imposed displacement histories were used . The findings provide information that is intended to help in the assessment of the joints. Figure 1 shows the test set-up.

Applicability

The results were obtained from tests conducted in 2000-2001. While every effort was made to use representative materials and configurations, logistical constraints and limited resources prevented exact simulations. In all the test specimens, the beams and columns were the same

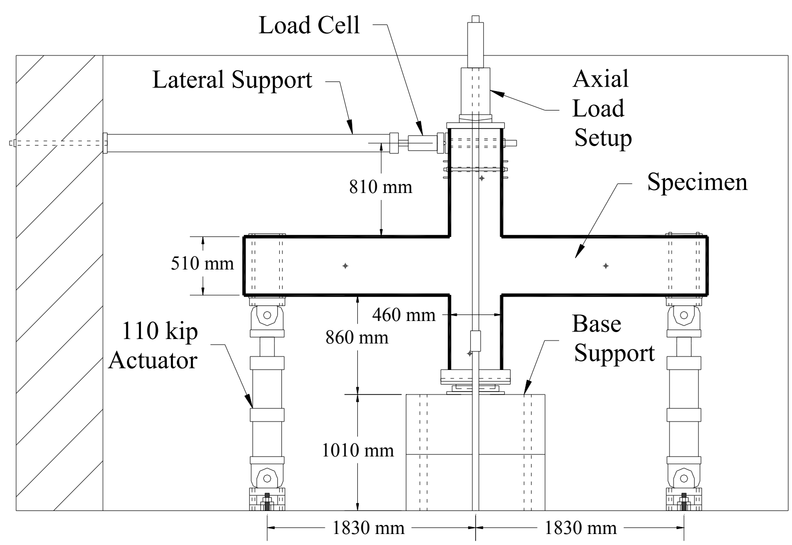

Figure 1. Test Set-up.

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width and were concentric. Modern deformed reinforcing bars, with a nominal yield strength of 60 ksi, were used, and the concrete strength was nominally 5000 psi, selected to represent the realistic strength today of concrete prior to about 1970. In each specimen, an axial load of $0.1f'_c A_g$ was applied to the column.

Findings from the Experiments

All the joints eventually suffered serious joint shear damage. The condition of a typical specimen at the end of the test is shown in Figure 2. Large quantities of concrete had been lost from the joint region and one or more the column bars was easily visible on each face.

Figure 2. Typical Joint at End of Test.

Damage was low, and easily repairable, prior to about 1.5% drift. Even when many load cycles to 1.5% drift were applied, the joint damage was modest. This raises the possibility of retrofitting a building by installing a stiff shear wall that will restrict the drift to less than 1.5% drift, and by leaving the joints themselves un-retrofitted. The low level of damage in the tests is in part attributed to the fact that the beam bars lost significant proportion of their bond strength almost as soon as they yielded, so they did not yield cyclically under an imposed cyclic drift of +/- 1.5%.

The shear strength envelope of the joints peaked at a drift of approximately 1.5% in all cases. The peak joint shear stress depended on the beam bars, which controlled the demand. After the peak, the shear stress dropped gradually to a value of approximately

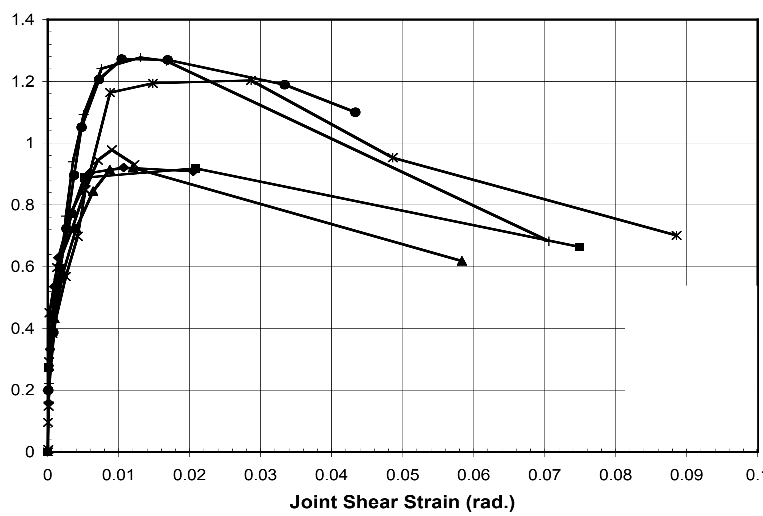

Figure 3. Joint Shear Stress-strain Envelopes.

8 f'_c (psi) at a drift of 5%, regardless of the beam bar strength. (Figure 3).

The FEMA 273 model for joint shear strength shows a strength that is consistent with the test results up to a joint shear strain of about 0.5%, after which it proves very conservative (Figure 4). The Joint Shear Factor is the joint shear stress divided by f'_c (psi).

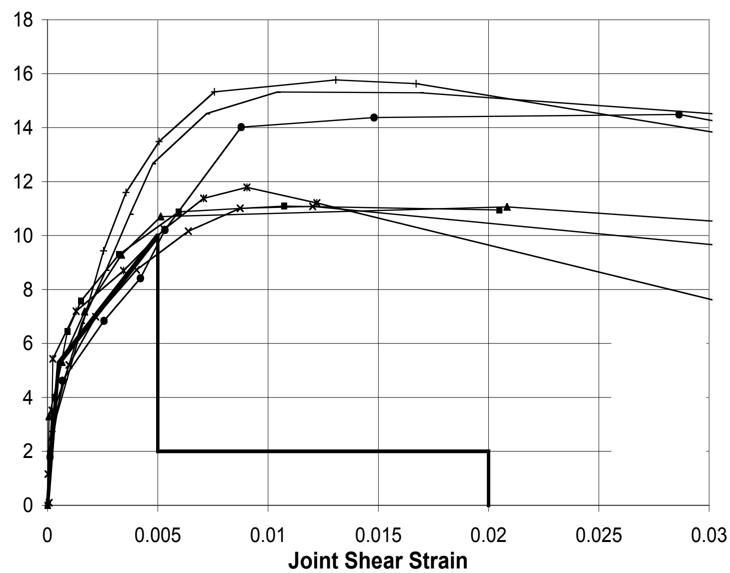

Figure 4. Comparison of Normalized Joint Shear stress-strain with Predictions of FEMA Model.

Joint shear strain contributes significantly to the total drift. In most of the specimens, the joint shear deformations were responsible for more than 60% of the total drift, after only three cycles of load. This means that, in the absence of other elements, such as a wall, to provide lateral strength and stiffness, the drift of such a frame will be significantly larger than that predicted if joint shear deformations are ignored. The latter is common practice today. The large drifts could lead to increased non-structural damage and, in extreme cases, to instability of the frame.

In all the specimens, the axial load of $0.1f'_c A_g$ was carried by the column without difficulty throughout the test. While this is encouraging, it should be noted that axial stresses in columns of the era prior to 1970 varied widely. Furthermore, in the tests, the axial capacity of the column bars at yield was at least 1.5 times the applied load. This provides a partial explanation for the good axial behavior.

For Further Information

The Walker et al. reference listed below provides further details. For additional information, please contact:

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Keywords
beam-column joints, reinforced concrete, joint shear, assessment, evaluation, damage, drift.