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BEHAVIOR OF SLAB-BAND FLOOR SYSTEMS SUBJECTED TO LATERAL EARTHQUAKE LOADING

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ABSTRACT

Results from an experimental investigation of the seismic behavior of wide-beam floor systems are reported. Six 3/4-scale reinforced concrete connection subassemblages were subjected to cycles of reversing lateral displacements, increasing in magnitude up to a maximum story drift of 5%. Test results indicate that wide-beam-to-column connections can be utilized in high seismic zones if appropriate design and detailing guidelines are followed. All specimens reached their full design strengths at drifts of between 1-1/2% and 2% and maintained their strengths up to drifts of 5%. Hysteresis loops of lateral load vs. lateral deflection for the connection subassemblages were moderately pinched, primarily due to some minor slippage of the wide beam and column bars at the connection. An analytical investigation of wide-beam structural frame systems is in progress.

KEYWORDS

Reinforced concrete; connections; wide-beam construction; quasi-static tests; seismic design; anchorage; shear; torsion; slab participation.

INTRODUCTION

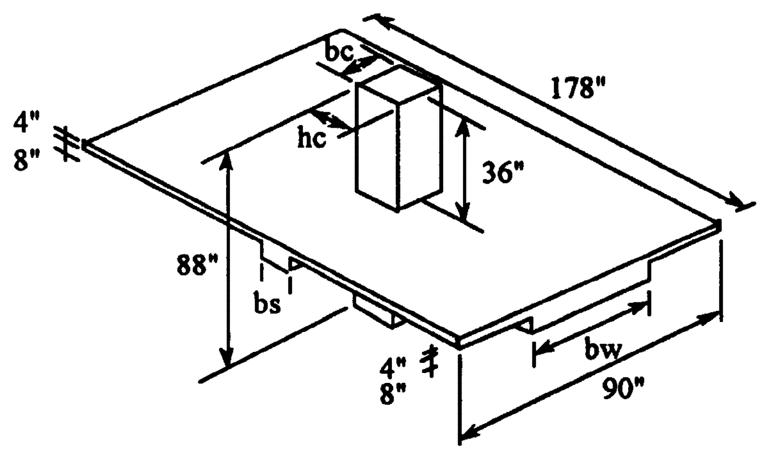
An investigation of the behavior of reinforced concrete (R/C) wide-beam-to-column connection subassemblages (with slabs) under seismic loading was initiated at the University of Michigan. This type of connection is found in a structural system that consists of floor slabs with wide and shallow beams framing into columns. It is frequently used in non-seismic regions because it simplifies formwork requirements and because overall building heights may be reduced for a given number of floors. The use of wide-beam construction in high seismic zones has the added benefit of reducing congestion of reinforcing steel in the connection region.

The use of wide-beam construction in high seismic zones is currently limited by both ACI 318-89 (ACI, 1989) and ACI 352R-91 (ACI, 1991). The main reason why the use of wide-beam construction is limited is because the cyclic behavior of connections with substantial amounts of the beam flexural steel anchored outside the column core is not known. It is feared that the wide beam and slab sections would not be able to develop and maintain their full design strengths and dissipate sufficient energy at reasonable levels of story drift due to anchorage problems, shear lag, and low stiffness of the wide beams. This study addresses these concerns for both interior and exterior wide beam-column-slab connections and develops appropriate design and detailing requirements for such connections. Analytical studies of wide-beam buildings are also in progress.

DESCRIPTION OF TEST SPECIMENS

The test specimens were approximately three-quarter scale R/C subassemblages representing wide beam-column-slab connections. Three interior connections and three exterior connections were tested. The subassemblages were terminated at approximately half of the column height above and below the floor and at approximately midspan of the beam. All specimens were designed and detailed in accordance with the requirements of the ACI Building Code and the recommendations of ACI 352R-91, except where specific parameters were allowed to deviate from recommended values for the purposes of the testing program. The design concrete compressive strength (f'c) was 4000 psi (27.6 MPa), and Grade 60 (nominal f_y = 60 ksi = 414 MPa) deformed reinforcing bars were used.

An overall view of the interior test specimens is given in Fig. 1. Figure 2 shows the beam and column cross-sections for interior specimen IWB-3.



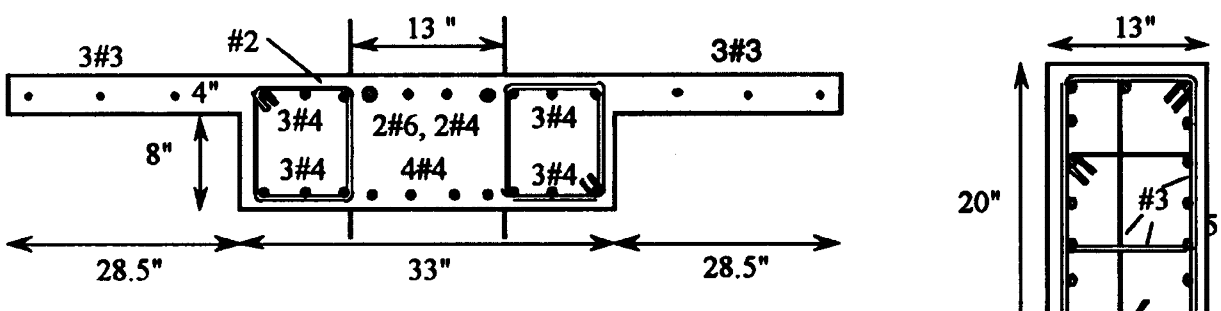
Variable Dimensions

	bc	hc	bw	bs
IWB1	14	14	35	12
IWB2	14	14	26	12
IWB3	13	20	33	16

Dimensions in inches

1 inch = 25.4 mm

Fig. 1. Interior Subassemblage Geometry



Wide Beam & Slab Section

Column Section

Note: U.S. reinforcing bar sizes; dimensions in inches, 1 inch = 25.4 mm

Fig. 2. Beam and Column Sections for Specimen IWB-3

An overall view of the exterior test specimens is given in Fig. 3. Figure 4 shows the beam and column cross-sections for exterior specimen EWB-3.

Variable Dimensions (in.)

	b_w	b_c	h_c	b_s	h_s
EWB-1	34	14	14	14	16
EWB-2	34	14	14	10	24
EWB-3	37	12	20	12	18

(1 inch = 25.4 mm)

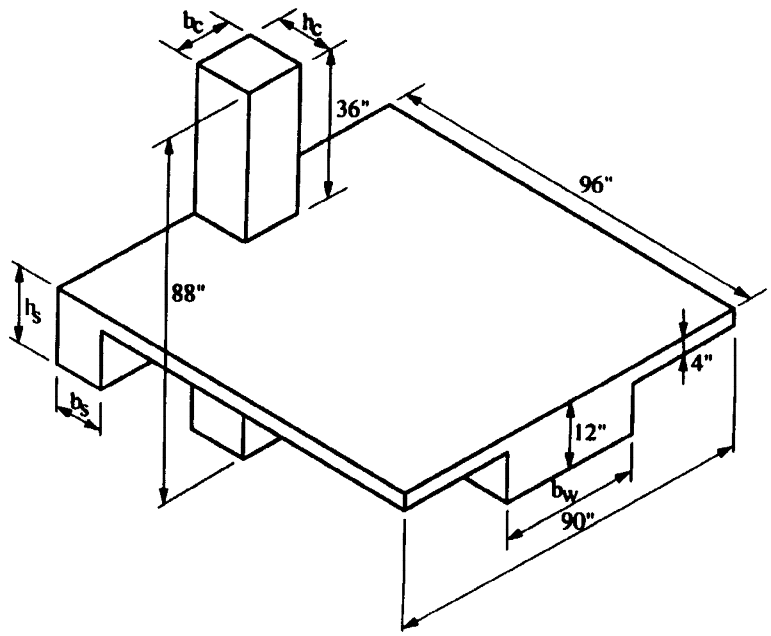


Fig. 3. Exterior Subassemblage Geometry

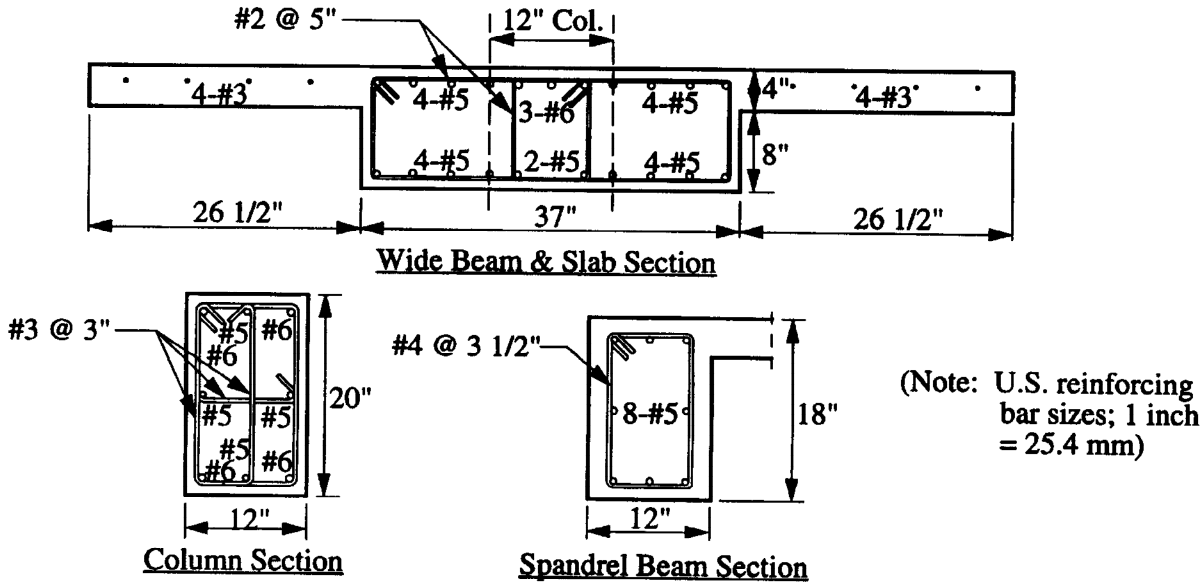


Fig. 4. Beam and Column Sections for Specimen EWB-3

The key design parameters for all six specimens are given in Table 1.

Two of the most important parameters studied were the wide-beam width to column-width ratio (b_w/b_c) and the percentage of wide-beam and slab flexural steel that was anchored inside the confined column core. In general, these two parameters are related, with larger b_w/b_c ratios leading to less flexural steel anchored inside the column core, but the distribution of steel across the width of the beam can alter the expected relationship between the two. The b_w/b_c ratio varied from 1.86 up to 3.08. The percentage of flexural steel anchored in the column core ranged from 20% to 52%. Both of these values would be equal or close to 1.0 for conventional R/C frame construction. Another important parameter in this study was the column aspect ratio, h_c/b_c, which varied from unity up to 1.67.

Table 1. Design Parameters For Subassemblages

Specimen	IWB-1	IWB-2	IWB-3	EWB-1	EWB-2	EWB-3
b_w/b_c	2.50	1.86	2.54	2.43	2.43	3.08
h_c/b_c	1.00	1.00	1.54	1.00	1.00	1.67
Top steel anchored in column core	39%	39%	44%	47%	51%	33%
Bottom steel anchored in column core	38%	38%	40%	44%	44%	20%
M_r $(=\Sigma M_{nc}/\Sigma M_{nb})$	1.20	1.25	1.50	1.60	1.45	1.45
γ $(= V/\sqrt{f'_c}\, b j h_c)^1$	17.0	20.0	17.0	14.5	16.0	18.0
h_b/d_b(col.)	16	16	19	16	16	16
h_c/d_b(beam)	22 & 28	22 & 28	27 & 40	-	-	-

1: psi units

The affect of different spacings of wide-beam shear reinforcement was also studied in these tests. Wide-beam stirrups were spaced at 3d/8 or at d/2 (where "d" is the effective depth of the beam) in all of the specimens. This was done as a result of the findings of a previous study on wide-beam construction, also performed at the University of Michigan (Gentry and Wight, 1994). Both of these spacings are greater than the code-specified spacing of d/4 for plastic hinge regions of beams that are in frames resisting seismic forces.

The ratio h_b/d_b (col), where " h_b" is the beam depth and "d_b(col)" is the column bar diameter, was relaxed to 16, based on a previous study (Gentry and Wight, 1994). The ratio of column depth to the diameter of the beam longitudinal bars in the interior specimens was kept above 20. Another parameter studied in the interior specimens was the confinement of the concrete outside the column core. Specimens IWB-(1 and 3) had closed hoops outside of the column section to improve the bond behavior of the beam longitudinal bars passing outside the column. Specimen IWB-2, whose width complied with the ACI (1989) width limit, did not have these side hoops. Finally, the spandrel beam depth, strength, and stiffness were varied in the exterior specimens.

TEST SETUP

The subassemblages were tested in the University of Michigan Structural Engineering Laboratory using the test setup shown in Fig. 5. A 50 k capacity actuator, reacting against the laboratory structural wall, was used to displace the top of the column horizontally through the quasi-static loading history also shown in Fig. 5, which was intended to simulate the effects of inelastic lateral loading due to an earthquake. No axial load was applied to the columns. The pins at the top, bottom, and ends of the specimens represent assumed locations of points of inflection. The displacement history ranged from cycles at 0.25% drift to cycles at 5% drift. The specimens were instrumented externally with load cells, LVDTs, and potentiometers. Also, approximately 50 electrical resistance strain gages were attached to the reinforcing steel at key locations inside each specimen.

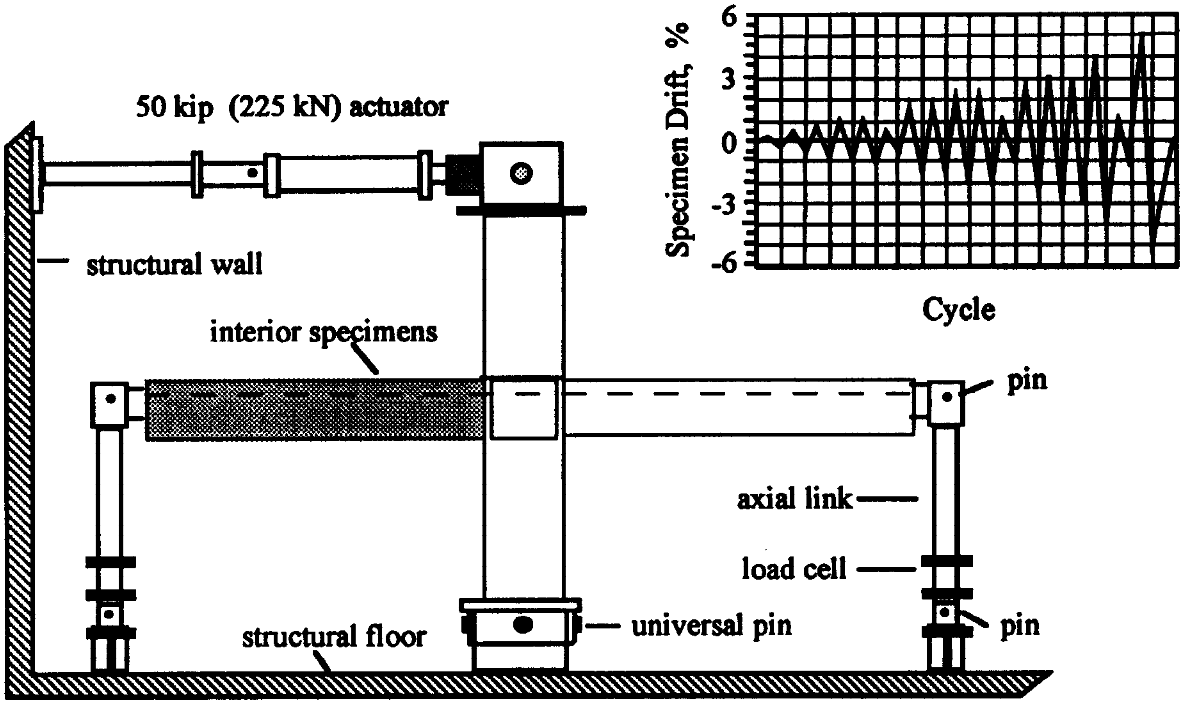


Fig. 5. Testing setup and loading routine

EXPERIMENTAL RESULTS

All of the specimens attained their negative and positive design strengths at subassembly drifts of between 1-1/2% and 2%. As drifts were increased, none of the specimens exhibited any significant drop in load capacity. All of the lateral load vs. displacement hysteresis loops were pinched to some degree. In the case of exterior connections, the loops became more "full" as the level of drift increased. Overall load-deflection plots for a typical interior specimen and a typical exterior specimen are given in Figs. 6 and 7, respectively.

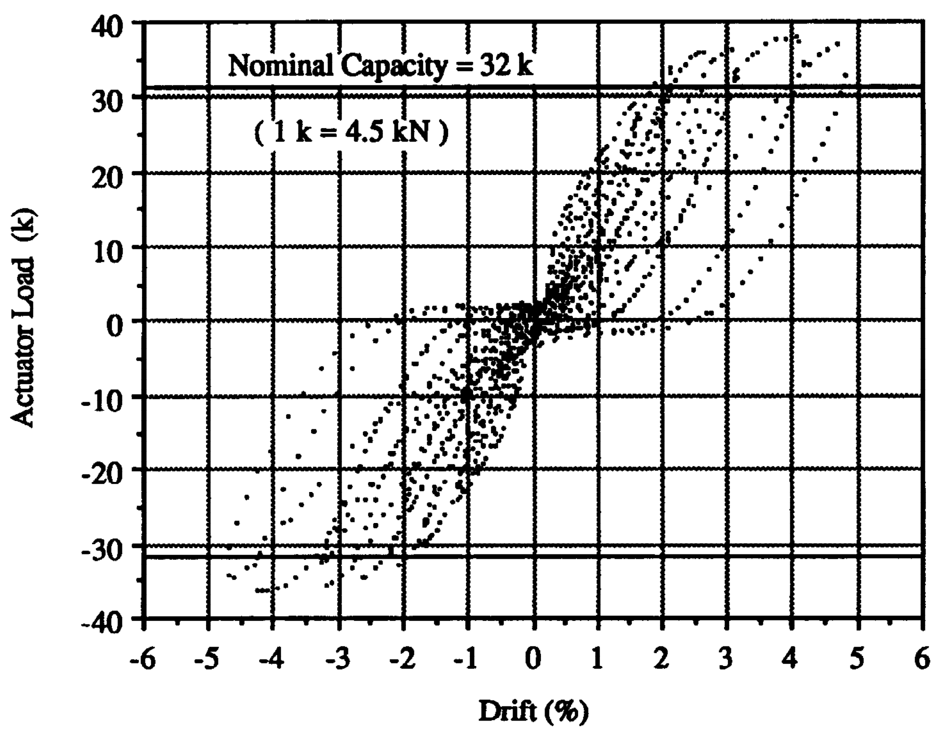


Fig. 6. Overall load-deflection relationship for Specimen IWB-3

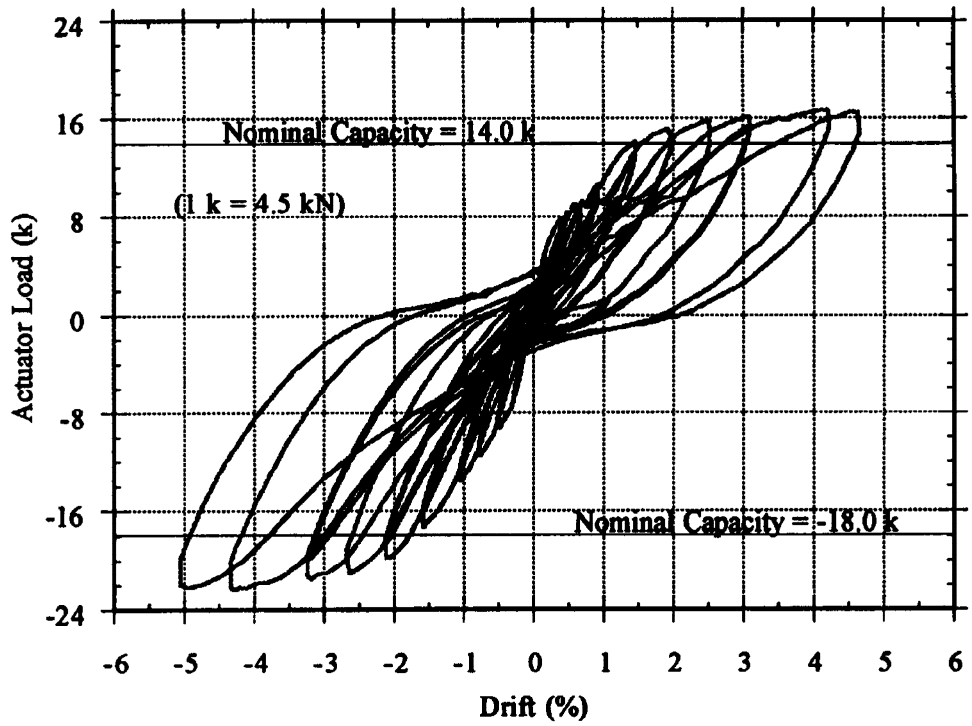
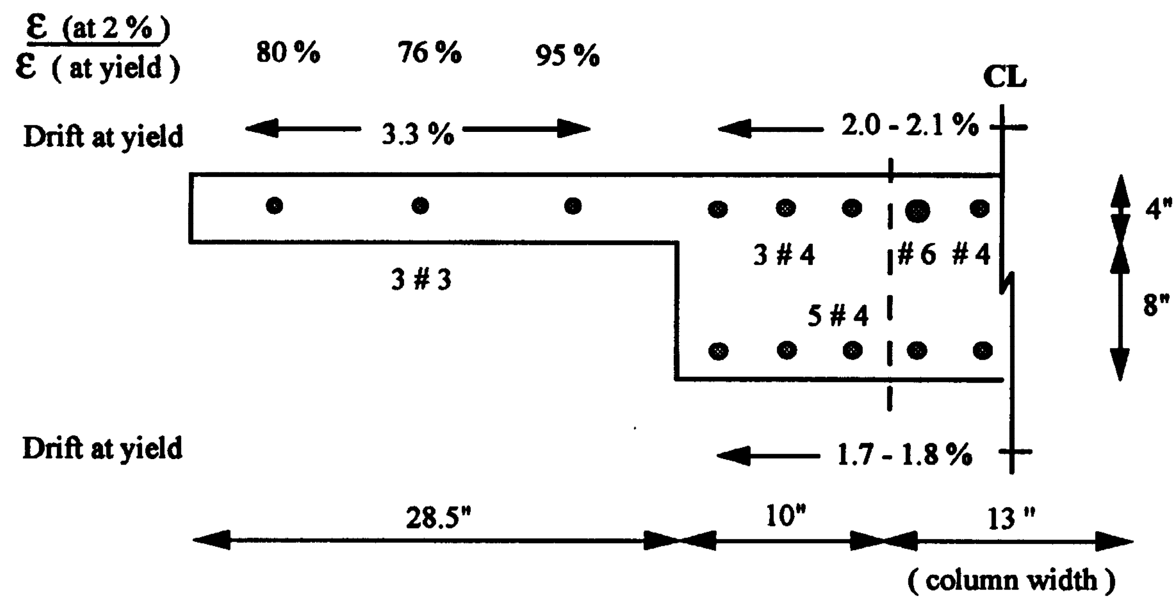


Fig. 7. Overall load-deflection relationship for Specimen EWB-2

Yielding of the wide-beam flexural steel anchored in the column core typically began during the first cycle to 1-1/2% drift. During the first cycle to 2% drift, reinforcing bars over the entire width of the wide beam had yielded. The transverse spread of the beam plastic hinge with increasing drift is shown in Fig. 8 for a typical interior specimen.



Note: U.S. reinforcing bar sizes; dimensions in inches; 1 inch = 25.4 mm

Fig. 8. Plastic hinge spreading for a typical interior specimen

In all specimens, the reinforcement in the slab participated during negative bending. In general, the level of slab participation increased as drift increased, and slab participation was greater for the case of deep, rectangular columns.

The specimens that used wide-beam shear reinforcement spaced at d/2 all performed well. None of the wide-beams had any inclined cracks and there was no tendency for the plastic hinge zones to break down under the reversed-cyclic loading.

Spandrel beam torsion played an important role in the behavior of the exterior specimens. Torsional cracking was found to be acceptable in the spandrel beam, but it was not acceptable for the longitudinal and transverse torsional reinforcement to yield. Yielding of torsion reinforcement in the spandrel beam permited excessively large rotations at the connection that prevented the full wide-beam moment from developing at drifts of 2%.

All interior specimens experienced an early loss of bond in the column bars, especially those specimens for which the h_b/d_b (col) ratio was relaxed. Wide beam bars passing outside the column core also showed some bond loss. This slippage was reduced by placing closed hoops adjacent to the columns. Slippage of the column and wide beam bars contributed to the pinching of the hysteretic loops. All the interior specimens were designed with moment strength ratios and joint shear capacities very close to the recommended limits. A more conservative design (larger moment ratio, smaller joint shear, and larger h/d_b ratios) would effectively reduce the pinching of the hysteretic loops for interior specimens.

WIDE BEAM FRAME BEHAVIOR

Analytical studies of wide beam buildings are currently underway. These studies are focused on the use of wide beam framing systems alone and in conjunction with perimeter frames or structural walls in regions of high and moderate seismicity. The reduced lateral stiffness of wide beam framing systems is being addressed in these studies.

CONCLUSIONS

Results from this experimental study indicate that wide-beam construction can be utilized in high seismic zones if appropriate design and detailing guidelines are followed. Preliminary results from an analytical investigation on wide-beam structural frame systems corroborate this finding.

All specimens tested reached their full design strength before 2% drift and maintained their strength up to drifts of 5%. The use of hoops adjacent to the columns provided effective confinement and prevented excessive slip of the wide beam bars passing outside the column core in interior connections. Shear reinforcement spaced at d/2 was found to be acceptable in the wide beam plastic hinge zones. Finally, for exterior connections, it was found that spandrel beam torsional cracking is acceptable, but yielding of the longitudinal and transverse torsional reinforcement is not acceptable.

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