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SLAB PARTICIPATION IN RC BUILDING LATERAL RESPONSE

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SUMMARY

After some recent damaging earthquakes, the United States and Japan started a Cooperative Research Program Utilizing Large-Scale Testing Facilities to obtain improved design guidelines for reinforced concrete (RC) structures. Large amounts of data collected from the seven-story building (1:1) tested at the Building Research Institute (BRI), Tsukuba, Japan, the (1:1) University of Texas at Austin (UTA) and the (1:12.5) Stanford University (SU) component tests, showed the slab importance in resisting lateral loads. The BRI building ultimate capacity under large drifts (1.5%) is evaluated based on the available test data.

INTRODUCTION

The 1981 test performed on the BRI building provided an excellent opportunity for the structural engineer to assess the performance and safety of a complete system which was designed based on equations derived mostly from isolated tests. Correlations between the BRI building critical regions, i.e., the connections zones between column, beam and slab, and the US component tests were established. In order to correctly evaluate the BRI building ultimate strength the Upper Bound Theorem was used in the virtual work computations. It included the consideration of 3-D effects, actual material properties, probable beam plastic hinge location and, the effective slab width monolithic with the beams.

SPECIMEN DETAILS AND MATERIAL PROPERTIES

The BRI building was tested under several different one-dimensional lateral load histories and the US components, reproducing the complete structure critical regions at level Z2, under beam tip loading, Fig. 1. The building material properties consisted of 4.1 ksi (28,5 MPa) concrete and Grade 50 reinforcement (f_{ys} = 53 ksi (360 MPa), f_{us} = 80 ksi (560 MPa). The UTA components had 4.8 ksi (33 MPa) concrete and Grade 60 reinforcement (f_{ys} = 60 ksi (420 MPa), f_{us}= 86 ksi (600 MPa)). Cross section dimensions, reinforcement details, instrumentation and, load histories are completely described Ref. 1.

BRI BUILDING ULTIMATE STRENGTH EVALUATION - EXPERIMENTAL EVIDENCE

Ultimate strength evaluation is used in seismic design because of the need

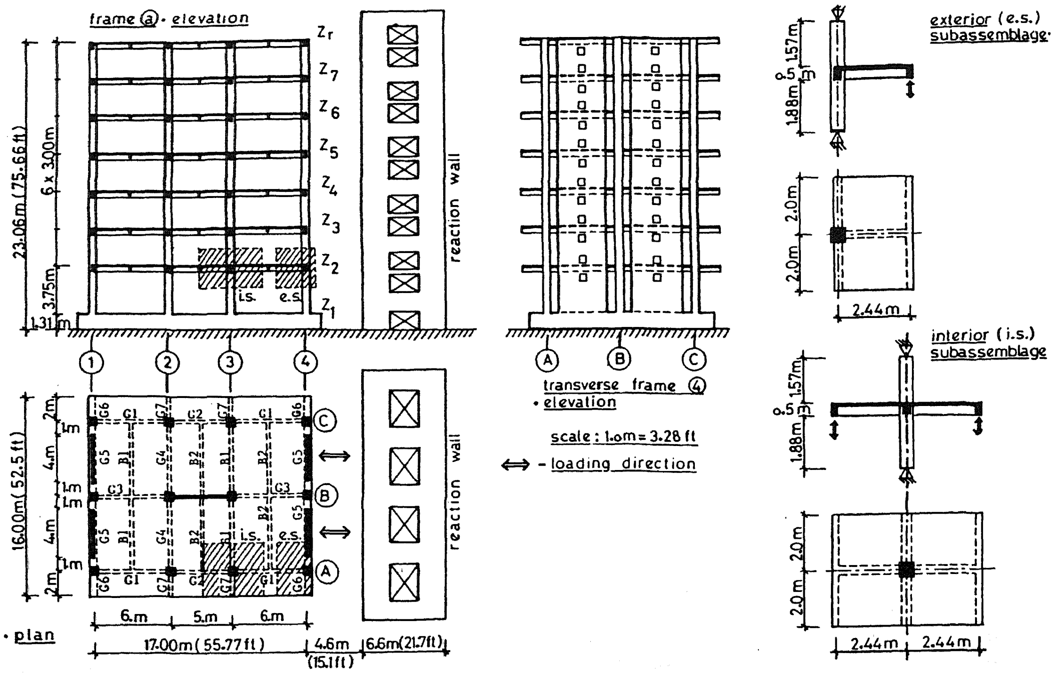


Fig. 1 BRI Building and UTA Components

to obtain simply and reliably the capacity of the structure. The deformation level attained in a RC frame under extreme lateral motion creates a characteristic beam plastic hinge pattern which is different from that under gravity loading. The spread of yielding, hinge location and overall frame deformation is closely related to the amount of imposed lateral drift.

At the end of test PSD-4, a maximum drift of $R_{max} = 1/64$ was attained in the BRI frame with generalized member yielding and a flat load-deflection response implying that the structure had reached its maximum capacity, Fig. 2 (Ref. 2).

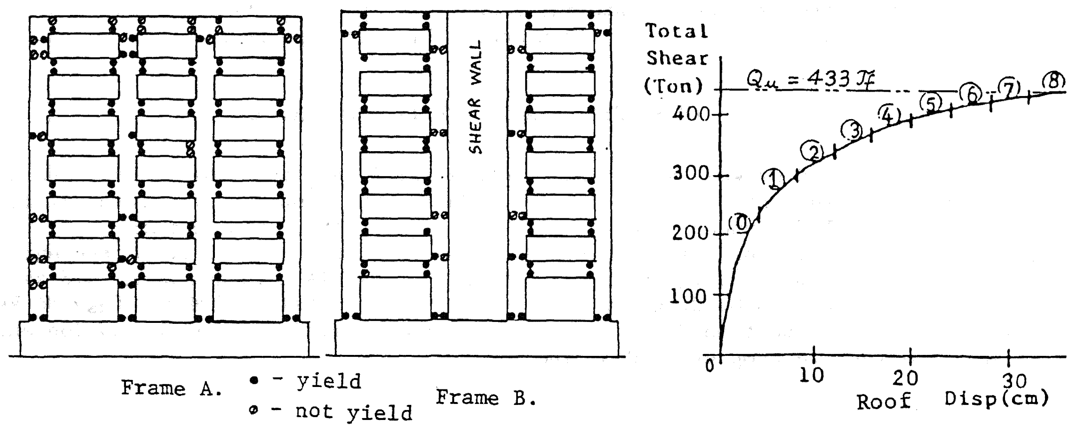


Fig. 2 BRI Building at the End of Test PSD-4

Under increasing displacement levels, the exterior joint floor strain distribution showed that the end walls imposed a restraint on the transverse beam. This action increased the yielding strains over the slab section at early loading stages. Near ultimate, yielding occurred over a large cross section portion in both building and components. Top slab strains in the BRI building overhang section remain low even at large displacements as compared to the UTA components because of different imposed loading or deformation and different boundary conditions, Fig. 3 (Ref. 1).

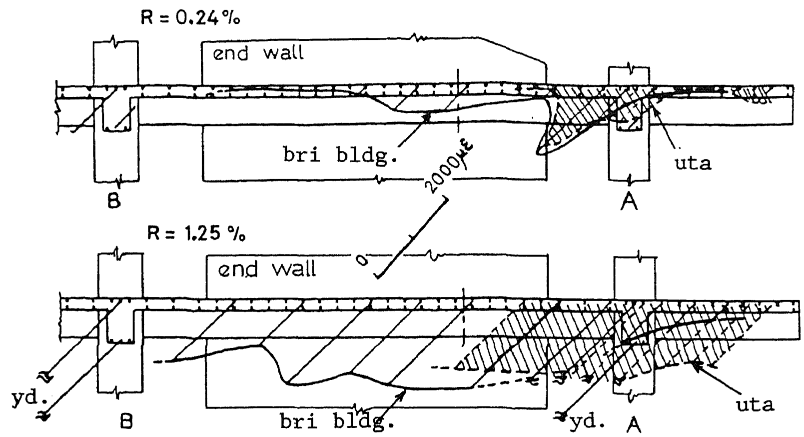


Fig. 3 Exterior Joint Slab Strain Distribution

The interior joint regions showed that spread of yielding occurred along the cross section in the lower slab bass at very large drifts, (Ref. 1). The observed beam rotations were similar to the imposed building drift when the slab was in compression, whereas when the slab was in tension they were 50% less than *the imposed drift*, (Ref. 2). A *plastic hinge length difference occurred depending* on the loading direction and the slab width in T-beam action. The shear wall uplift effect on Frame B duplicated the amount of the beam end rotation, Fig. 4.

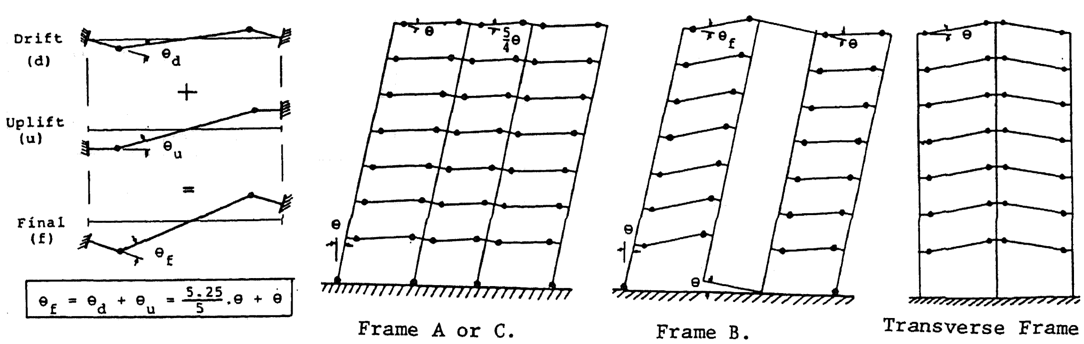


Fig. 4 Assumed Failure Mode in the BRI Building

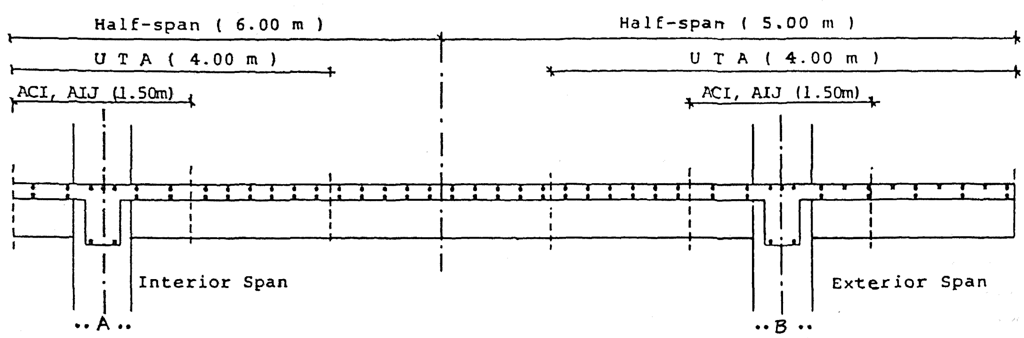
Shear wall deformation studies showed that most of the wall curvature, rotation was located in the lower 2/3 of level Z2 for R_{max} = 1/64 (Ref. 2). Based on the recorded data, the possible BRI building failure mechanism is displayed in Fig.4.

BRI BUILDING ULTIMATE STRENGTH

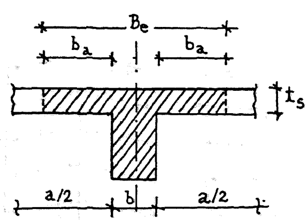
The Upper Bound Theorem for limit analysis was used in the virtual work computations to evaluate the ultimate capacity. An inverted triangular loading pattern along the building height developed the assumed failure mechanism. The total external work, W_e, delivered by the actuators is given in terms of the horizontal base shear, V_b, by:

$$W_e = 613.V_b.\theta \qquad \text{(kips-in.)} \tag{1}$$

where θ = drift rotation. The internal work contribution was studied using four different slab widths and was compared with the measured base shear, V_{exp}= 433 tf (954 k.) attained at R_{max} = 1/64. The four slab widths considered were: (1) bare frame-rectangular beam; (2) ACI, AIJ 1.5 m (59 in.); (3) UTA 4.0 m (157.5 in.); and, (4) half-span - 5.0 m (197. in.) or 6.0 m (236 in.). In case (2), both Codes (Refs. 3,4) coincide on the effective slab width for the BRI frame, Fig. 5. These Code provisions were developed based on tests with the slab in compression. As the Codes omit provisions for the case of slab in tension designers have been using positive moment effective slab width for both cases.



a. Different Effective Slab Widths



ACI 318-83 /8.10.2	Span 6.00 m
1. $B_e \le \frac{L_{bm}}{4}$	1.50 m
2. $B_e \le b + 2 \times 8t_s$	2.22 m
3. $B_e \le \frac{a}{2}$	3.00 m

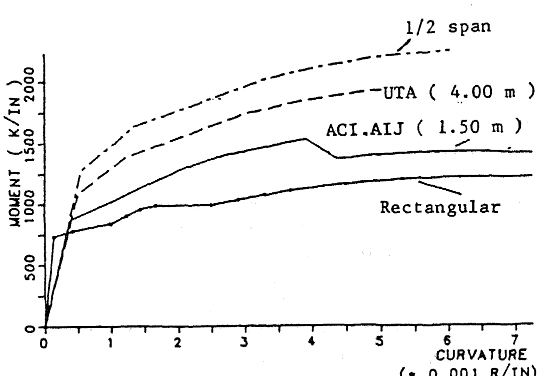
AIJ - 82 / 8.(3)	Span 6.00 m
1. $b_a = (\frac{1}{2} - 0.6\frac{a}{l})$ $a < 1/2$	$a > 1/2$ $\therefore$ $b_a = 0.60$ m
2. $b_a = 0.1\ l$ $a > 1/2$	$B_e = 1.50$ m

b. ACI, AIJ Code Effective Slab Widths

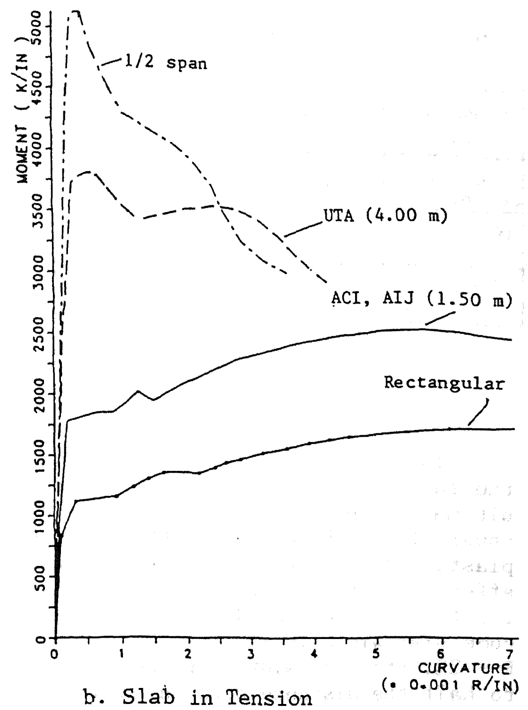
Fig. 5 Effective Slab Width in BRI Computations

The shear wall plastic hinge was assumed to be at the column axis although the actual location was near the shear wall - boundary column interface.

Each structural member (beams, columns, shear wall) moment-curvature properties was studied with the RCCOLA program developed at the University of California Berkeley and modified by Farahany et al. (Ref. 5), Fig. 6.



a. Slab in Compression



b. Slab in Tension

Fig. 6 Longitudinal Beam Moment Curvature Characteristics

If the average member rotation, θ_{av}, is proportional to the building drift, the average member curvature, ϕ_{av}, over a certain plastic hinge length, L_p, can be easily found:

$$\phi_{av} = \theta_{av} / L_p \tag{2}$$

The internal work, W_i, was calculated with beam rotation at d/2 (d = member depth), (Ref. 1):

$$W_i = 14[M_t(+) + M_t(-)]\theta + M_s\theta + 21[M_i(+) + M_i(-)]\theta + 26[M_e(+)+ M_e(-)]\theta$$
$$+ 12[M_e(+) + M_e(-)](1.25\ \theta) + [10\ M_c(b) + 4\ M_c(t)]\theta \tag{3}$$

where the plastic moments are: M_t = transverse frame beam; M_s = shear wall; M_i = interior Frame B longitudinal beam; M_e = exterior Frames A and C longitudinal beams; M_c = column. The bracketed indices correspond to: (+) positive and (-) negative moment; (t) top and (b) bottom of column. In the shear wall, the "prestressing" effect developed by the transverse frame with an axial load increase was also taken into account in the virtual work computations, (Ref. 1). The beam hinging location choice at the column axis, column face and at a distance d/2 from column face is shown in Table 1.

At R = 1/64, the column and transverse beam plastic hinges were responsible for 20% of the total base shear. The shear wall participation decreased from 42% to 28%, if larger amounts of effective slab width were assigned to the beams from bare frame to half-span widths. If the total slab width was considered, Frame B resisted nearly 50% of the ultimate base shear, Frames A and C, 20% each, and the transverse frame, 10%, Table 2.

Table 1 Different Beam Hinge Location Effects V_{calc}/V_{exp}

Beam Pl. Hinge Loc.	Bare Frame	ACI, AIJ (1.5m)	UTA (4 m)	Half-span
@ Column centerline	0.56	0.67	0.86	0.94
@ Column face	0.58	0.70	0.91	1.00
@ d/2 from col. face	0.61	0.73	0.97	1.06

Table 2 Member Participation in Ultimate Strength

Partic. (%)	Trv. Frame	Frame B			Frame A & C	Cols.
		Sh.wall	Beam	Σ		
1. Bare Frame	7.7	42.3	12.6	54.4	24.3	13.1
2. ACI, AIJ	8.6	37.3	14.9	52.2	28.7	10.6
3. UTA	9.5	29.5	18.1	47.6	34.8	8.0
4. Half-span	10.2	27.6	20.0	47.6	35.0	7.3

CONCLUSIONS

The ultimate BRI building strength was evaluated upon an educated guess on the failure mechanism. At a very large drift of 1.56%, the computed BRI building ultimate moment capacity had better agreement with the experimental measured base shear if the whole slab width was considered effective with the beams, the beam plastic hinge location was at d/2 from the column face, and the transverse frame effect over the shear wall was also considered. Consequently, it is proposed that for this type of buildings up to deformation levels R = 0.5% the current design code slab width is satisfactory. However, near ultimate and for large drifts of R = 2%, the slab width working monolithically with the beam had to be increased to half the distance between adjacent frames.

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