



DEFICIENCIES IN BRACING OF LIFT SLAB BUILDINGS

Lift slab construction is a method of erecting post-tensioned reinforced concrete floor and roof slabs which have been constructed on the ground. Although there are various different methods, the one that has been used most in the United States involves steel columns, jacks placed on top of the columns to lift the slabs, and threaded lifting rods connected to the jacks and to steel lifting collars cast into each of the concrete slabs. See Figure 1.

The unique features of lift slab buildings outside of the fact that the slabs are lifted into place are the lifting collars which also act as shear heads^{1,2} to transfer forces from the slabs to the columns, and the connections of the lifting collars to the steel columns. The support of the lifting collars is provided by wedges bearing on the edges of plates (called *weld blocks*) which are welded to the flanges of the columns as shown on Figures 2 and 3 (on page 2). There are two connections on each column, one on each flange. Figure 2 shows a slab temporarily parked. When the slab reaches its final position, the wedges are welded to the lifting collar, column, and weld block, as shown on Figures 3 and 4.

The lateral load resisting system of lift slab buildings usually consists of reinforced concrete shear walls,³ or braced frames with steel diagonal members connected to the steel columns. Although the just described connections of the slabs to the columns can provide some moment resistance, this resistance is very limited unless there is special design and detailing of the joints.



Figure 1 Lift slab construction of hotel tower in Portland, ME. Lifting is completed for the first tier of columns; second tier of columns with jacks on top are partially erected; slabs at top are temporarily parked.

Improper Bracing of Hotel Tower

Figure 1 shows a circular hotel guest room tower when it was under construction, in Portland, Maine. Zallen Engineering was retained to make a structural investigation when concrete tests showed that the strength of the concrete in one of the post-tensioned floor slabs was materially deficient. The investigation revealed that the lateral load resisting system for the structure had substantial deficiencies.

Figure 5 shows a plan of a typical concrete floor, and Figure 6 shows a radial cross-section at a braced

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Figure 2 Detail at bottom of steel lifting collar. Slab temporarily parked. One of two lifting rods is shown on left; other rod is hidden on other side of column. Left and right sides of lifting collar extend beyond opening at bottom of slab, but extensions are hidden by cement paste.

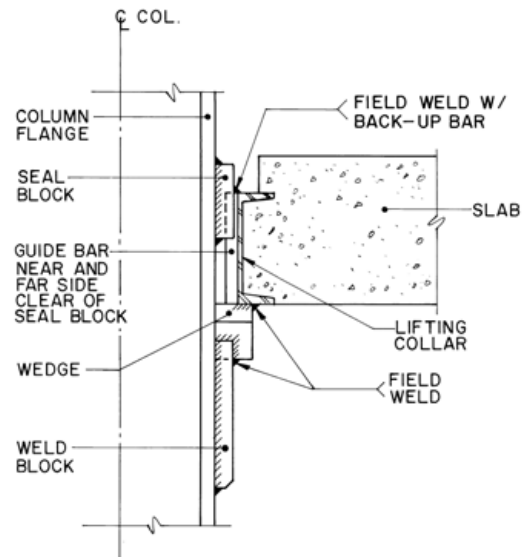


Figure 3 Section through lifting collar and column flange showing connections of lifting collar to the column flange. Seal block was omitted at multi-story condominium buildings.

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frame. The first deficiency was that the planes of all the brace frames were radial and intersected at a single vertical axis (at the center of the circle). There was no other *significant* lateral load resistance; thus the structure was torsionally unstable -- i.e. the whole structure could rotate about a vertical axis located at the center of the circle.

A second deficiency was that one end of the diagonal cross-bracing members did not terminate at a column, but terminated within the span of the slabs. See Figures 6 and 7. Under lateral load, the forces from the bracing would impose large vertical concentrated loads on the slabs which in turn would impose large bending effects (moments) on the slabs, which the slabs were not designed to resist.

A third deficiency was that the diagonal bracing was designed for strength but not stiffness, and a low 10 psf wind load, which resulted in very small sizes for the diagonal bracing. The resulting system is very flexible and substantially under strength; under a wind load, the (vertical) gravity loads acting on the structure in its *laterally deflected* position would amplify the effects of the wind load on the bracing by 36% (the P-) effect).

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Notes:

1. The notes explain structural engineering jargon for non-engineers
2. Shear is an internal force which, at any location along a member (any section), tries to push the part of the member on one side of the section past the part of the member on the other side of the section. Shear heads transfer shear (or load) between the slabs and the columns.
3. A shear wall resists lateral load in its own plane; it acts as a deep vertical cantilever beam.
4. The force causing or resisting rotation is defined as a moment or a bending moment. If the joints between the columns and the slabs have moment resistance, so that there is no relative rotation between columns and slabs at their connections, the system is called a rigid frame. The rigid joints prevent the rigid frame from swaying in its own plane under lateral load, which it would do if it had pinned joints.

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Zallen Engineering recommended that chevron type steel braced frames be constructed at strategic locations on the outer circumferential column line to provide the necessary lateral resistance for the building. However the architect and his structural engineer opted to create rigid frames in the circumferential direction, by reinforcing the connections between the slabs and the columns to obtain the required bending moment resistance.

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Figure 4 Wedges welded to weld block, column, and lifting collar.

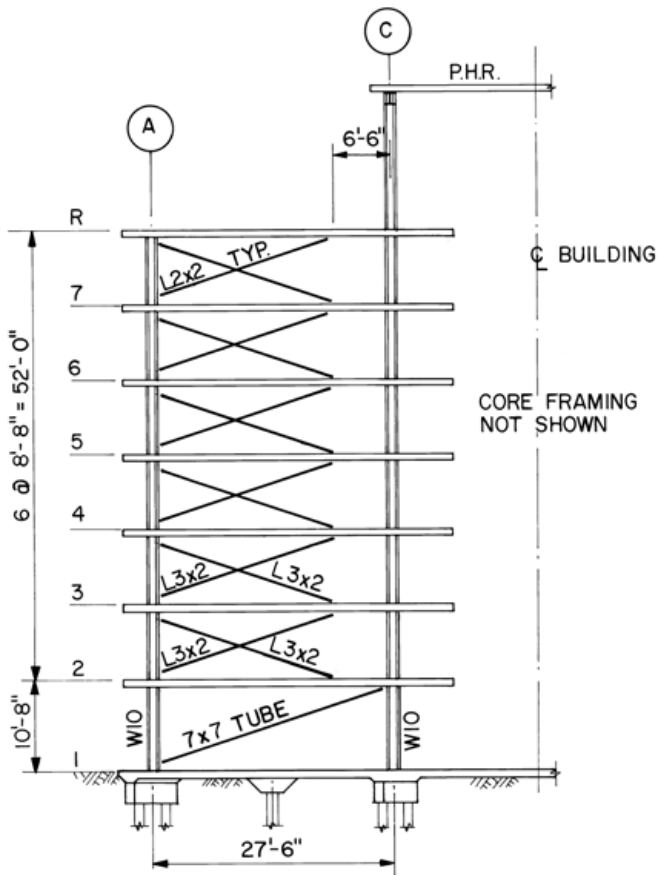


Figure 6 Schematic radial cross-section of hotel tower at a typical braced frame. Framing for core of building is not shown.

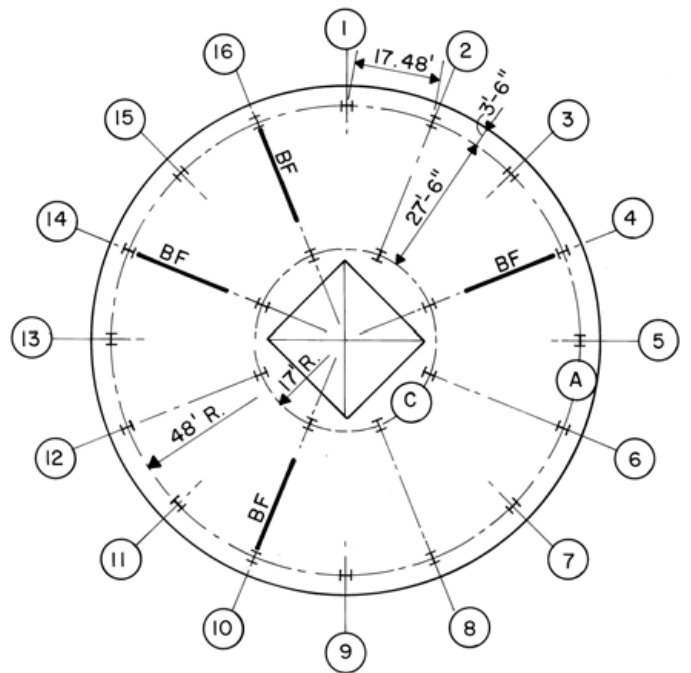


Figure 5 Schematic plan of typical floor of hotel tower. The mark BF denotes a radial braced frame. Framing for core of building is not shown.



Figure 7 Typical radial braced frame.

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Improper Bracing of Multi-Story Residential Condominium Buildings

A second example of improper bracing was the lateral load resisting system for a large condominium in suburban Boston. Zallen Engineering was retained to give a second opinion on the lateral load resisting system of the condominium buildings. The complex consists of 3 buildings separated from each other by expansion joints. There are seven stories of residential condominium units and a basement story garage. Figure 8 shows a general view of the buildings. Figure 9 shows a schematic floor plan. Figure 10 is a schematic cross-section through the buildings.



Figure 8 Residential condominium complex in suburban Boston.

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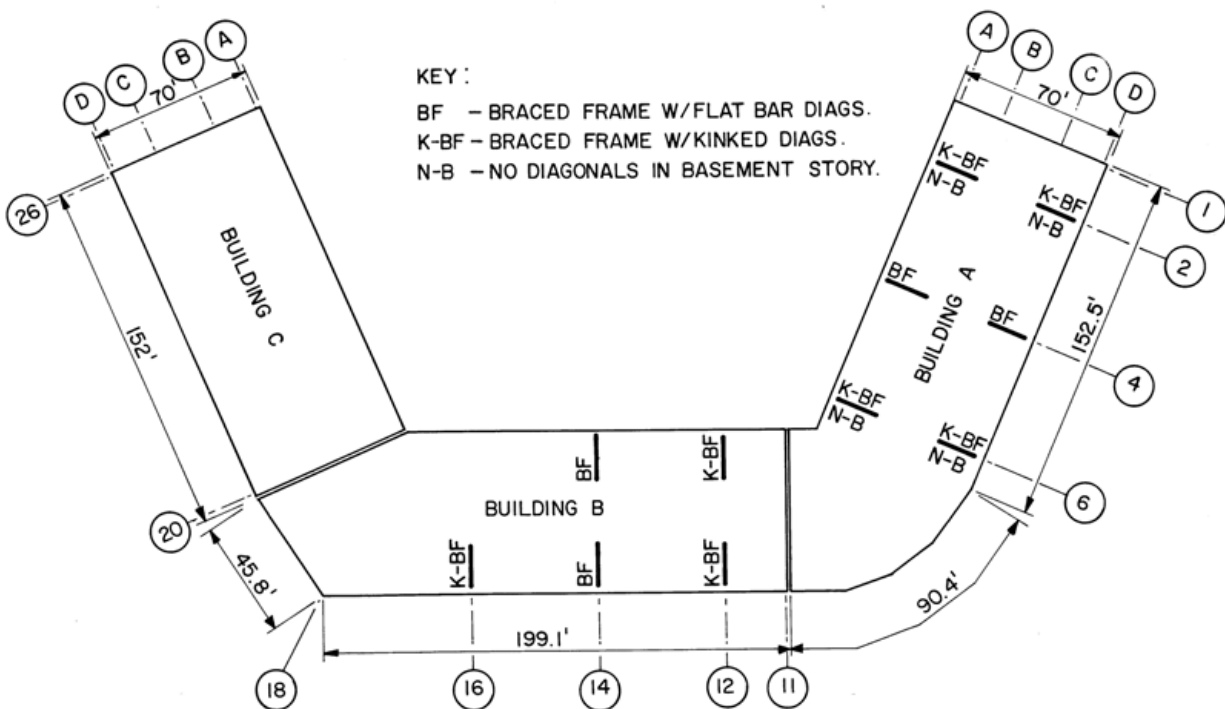


Figure 9 Schematic plan of typical floor of condominium complex, showing locations of braced frames.

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The lateral load resisting system consisted of braced frames from the basement to the bottom of the 4th floor, as located on Figure 9, and moment resisting (rigid) frames composed of floor slabs and columns where special connections of the lifting collars were made to *some* of the steel columns.

The braced frames had diagonals that could transmit tension only: straight flat bars, as shown on Figure 11, or a kinked configuration as shown on Figure 12. The location of each type is given on Figure 9. Referring to Figure 10, for wind in one direction, only the diagonals on one side of the building are effective, and for wind in the other direction, only the diagonals on the other side of the building are effective. As indicated on Figure 9, some of the diagonals were omitted from the basement story garage, and in one case, on Line 16, there was a braced frame on only one side of the building, which meant that the bracing on that line could resist lateral load in one direction only. Also, as indicated on Figure 9, there were no braced frames at all in Building C.

Referring to Figure 3, the seal blocks were omitted on this project, and at some locations which were designated to have special moment connections, the observed width of spaces between the column flanges and the lifting collars varied between $1\frac{1}{2}$ and $1\frac{7}{8}$ inches. At some of these locations, round bars were placed between the guide bars and column flanges, and were welded to each; at other locations, large round bars or two smaller bars welded to each other were used between the lifting collar and column flanges, and were welded to each; and in one observed case, $1/8$ inch diagonal plates covered the gap between the guide bars and the column flanges and were welded to each.

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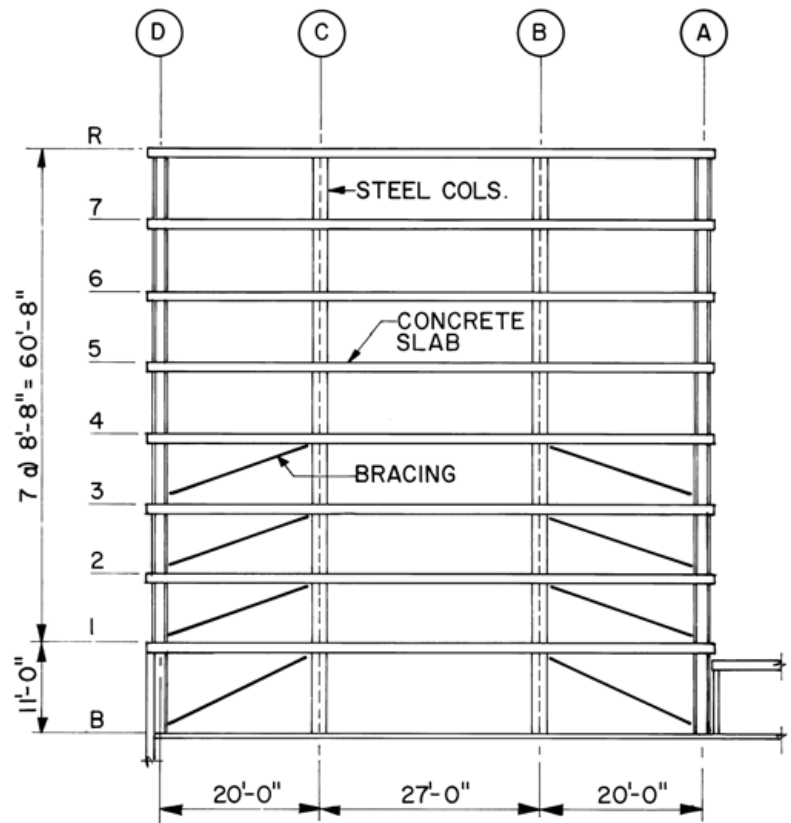


Figure 10 Schematic cross-section of condominium buildings showing braced frames with flat bar tension diagonals. At some locations the diagonals were "kinked" - see Figure 12. At some locations in Building A, basement story diagonals were omitted.



Figure 11 Typical flat bar diagonals of braced frames in condominium buildings; diagonals shown are in basement story of Building B. Columns are steel, but are encased in concrete in the basement garage.

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The lateral load resisting system was significantly overloaded when subjected to the code specified wind load. The kinked diagonals in the braced frames were essentially ineffective in resisting force and the moment connections between the slabs and the columns had little moment resistance and were generally inadequate. Further, some of the flat bar diagonals of the braced frames were severely overloaded, with ratios of load to safe load as high as 1.7, the nominal safety factor for steel.

The Massachusetts State Building Code required that the building be designed for seismic (earthquake) resistance. The lateral load resisting system was grossly overloaded when subjected to the code specified seismic loads, and moment resisting connections between flat slabs and columns are prohibited by the Code for seismic resistance, due to their lack of ductility.

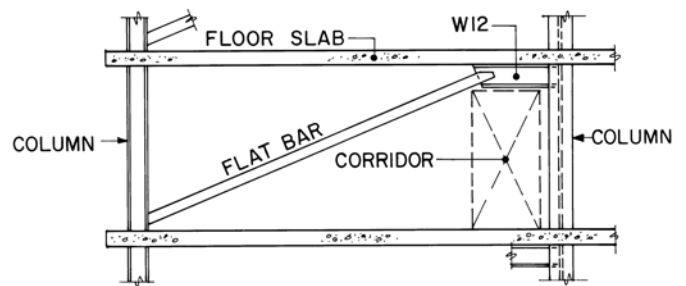


Figure 12 Typical kinked tension diagonal of braced frames.

Principal Rubin M. Zallen, P.E. investigated these bracing problems. He is the principal author of the book *Engineering Considerations for Lift-Slab Construction*, published by the American Society of Civil Engineers in 2004. Mr. Zallen is a member of the Task Committee on Lift-Slab Construction of the Technical Council on Forensic Engineering of the American Society of Civil Engineers.

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