L'Ambiance Plaza Collapse

The death of 28 workers in the April 23, 1987, construction collapse of the L'Ambiance Plaza building in Bridgeport, Connecticut, triggered a massive rescue effort and several investigations. Unfortunately, to this day the true cause of the collapse remains in dispute because a settlement ended all investigations. This was a lift-slab project.

In his textbook *Prestressed Concrete: A Fundamental Approach*, Nawy discusses lift-slab construction. He points out the need to keep the slabs level during lifting operations and notes that "the construction technique in lift slabs and the absence of the expertise required for such construction can create hazardous conditions which may result in loss of stability and structural collapse" (Nawy 2006, p. 556).

Design and Construction

L'Ambiance Plaza was planned to be a 16-story building with 13 apartment levels topping 3 parking levels. It consisted of two offset rectangular towers, 19.2 × 34.1 m (63 × 112 ft) each, connected by an elevator (Figs. 4-9 and 4-10). Post-tensioned concrete slabs 178 mm (7 in.) thick and steel columns made up its structural frame (Cuoco et al. 1992).

Post-tensioning overcomes the tensile weakness of concrete slabs by placing high-strength steel wires along their length or width before the concrete is poured. After the concrete hardens, hydraulic jacks pull and anchor the wires or strand, compressing the concrete (Levy and Salvadori 1992).

Using the lift-slab method, the floor slabs for all 16 levels were constructed on the ground, one on top of the other, with bond breakers between them (Fig. 4-9a). Then packages of two or three slabs were lifted into temporary position by a hydraulic lifting apparatus and held in place by steel wedges. The lifting apparatus consisted of a hydraulic jack on top of each column, with a pair of lifting rods extending down to lifting collars cast in the slab (Figs. 4-9b and 4-9c).

Once the slabs were positioned, they were permanently attached to the steel columns. Two shear walls in each tower were meant to provide the lateral resistance for the completed building on all but the top two floors. These two floors depended on the rigid joints between the steel columns and the concrete slabs for their stability. Because the shear wall played such an indispensable role in the lateral stability of the building, the structural drawings specified that during construction the shear walls should be within three floors of the lifted slabs (Heger 1991).

Figure 4-9. L'Ambiance Plaza Lift Slab Construction.
Courtesy National Institute of Standards and Technology.

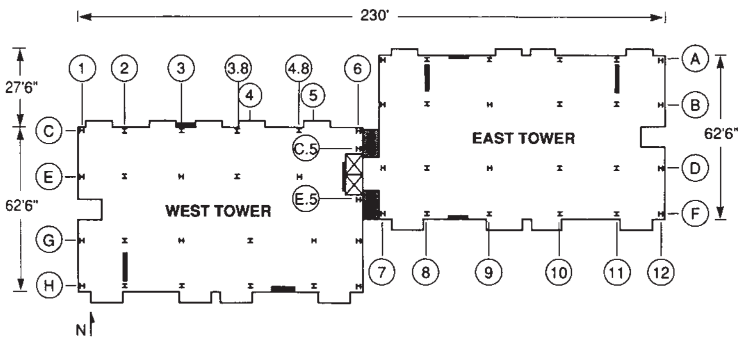

Figure 4-10. Floor plan of L'Ambiance Plaza.
Source: Moncarz et al. (1992).

Details of the lift-slab system, the competing lift-plate system, and other similar systems are provided by Zallen and Peraza (2004, pp. 7–21). The systems are proprietary.

Collapse

At the time of the collapse, the building was a little more than halfway completed. In the west tower, the 9th, 10th, and 11th floor slab package was parked in stage IV directly under the 12th floor and roof package (Fig. 4-11). The shear walls were about five levels below the lifted slabs (Cuoco et al. 1992).

The workers were tack-welding wedges under the 9th-to-11th floor package to temporarily hold them in position, when a loud metallic sound followed by rumbling was heard. Kenneth Shepard, an ironworker who was installing wedges at the time, looked up to see the slab over him "cracking like ice breaking." Suddenly, the slab fell onto the slab below it, which was unable to support this added weight and fell in turn. The entire structure collapsed, first the west tower and then the east tower, in 5 s, only 2.5 s longer than it would have taken an object to free fall from that height. Ten days of frantic rescue operations revealed that 28 construction workers had died in the collapse, making it the worst lift-slab construction accident ever. Kenneth Shepard was the only one on his crew to survive (Levy and Salvadori 1992). The collapsed structure is shown in Fig. 4-12.

Causes of Failure

All of the parties involved in the design and construction of the building hired forensic engineering firms to investigate possible causes of the

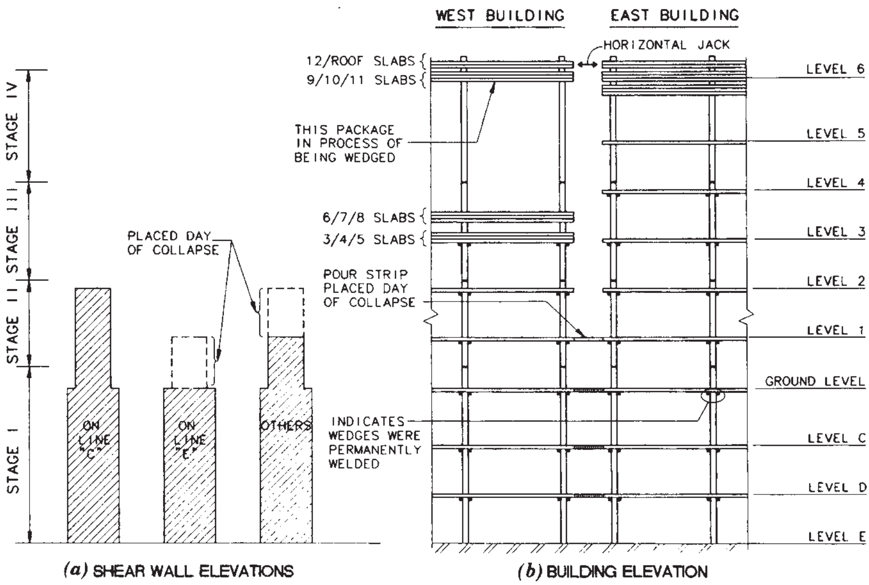

Figure 4-11. Elevation of L'Ambiance Plaza just before collapse.
Source: Cuoco et al. (1992).

failure. However, a prompt legal settlement prematurely ended all investigations of the collapse. Consequently, the exact cause of the collapse has never been established. The building had a number of deficiencies, any one of which could have triggered the collapse. The question, however, remains which one of these problems was in fact the triggering mechanism. There are six competing theories. Kaminetzky lists, but does not discuss, a seventh theory: "failure resulting from lateral soil pressure acting on the foundation walls" (Kaminetzky 1991, p. 82).

- Theory 1, National Bureau of Standards (NBS), now the National Institute of Standards and Technology (NIST): An overloaded steel angle welded to a shear head arm-channel deformed, causing the jack rod and lifting nut to slip out and the collapse to begin (Korman 1987).
- Theory 2, Thornton-Tomasetti Engineers (T-T): The instability of the wedges holding the 12th floor–roof package caused the collapse (Cuoco et al. 1992).
- Theory 3, Schupack Suarez Engineers, Inc. (SSE): The improper design of the post-tensioning tendons caused the collapse (Poston et al. 1991).

A

B

Figure 4-12a, b. The collapsed structure of L'Ambiance Plaza.
Courtesy National Institute of Standards and Technology.

- Theory 4, Occupational Safety and Health Administration (OSHA): Questionable weld details and substandard welds caused the collapse (McGuire 1992).
- Theory 5, Failure Analysis Associates, Inc. (FaAA): The sensitivity of L'Ambiance Plaza to lateral displacement caused its collapse through global instability (Moncarz et al. 1992).
- Theory 6, Oswald Rendon-Herrero: Rapid slump of a column footing precipitated the collapse (Rendon-Herrero 1994).

Theory 1—Overloaded Steel Angle

The NBS investigation concluded that the failure occurred at the building's most heavily loaded column, E4.8, or the adjacent column, E3.8, as a result of a lifting assembly failure (Fig. 4-13). The shear head reinforced the concrete slab at each column, transferred vertical loads from the slabs to the columns, and provided a place of attachment for the lifting assembly. It consisted of steel channels cast in the concrete slab, leaving a space for the lifting angle. The lifting angle had holes to pass the lifting rods through. These rods were raised by the hydraulic jacks on the columns above them (Levy and Salvadori 1992).

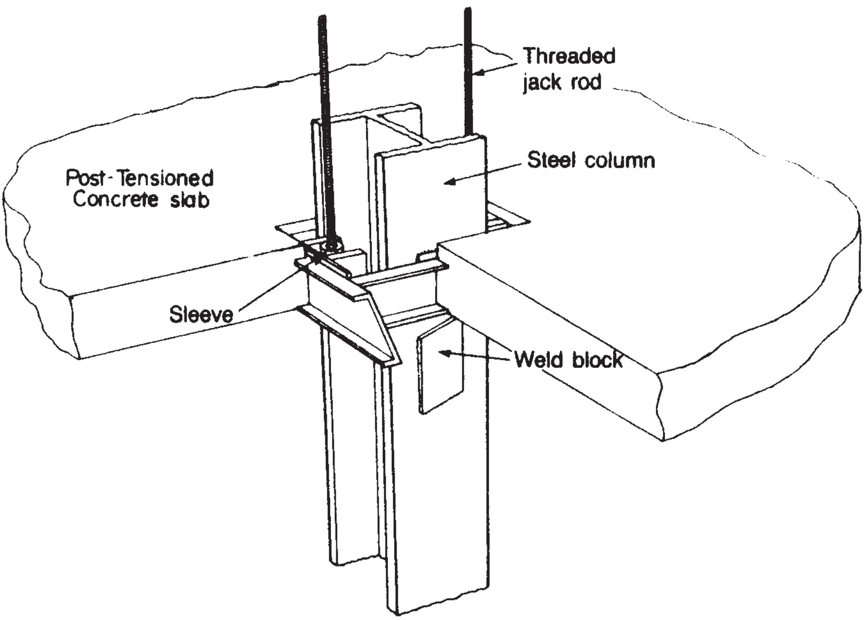

Figure 4-13a. Lifting assembly.
Source: Poston et al. (1991).

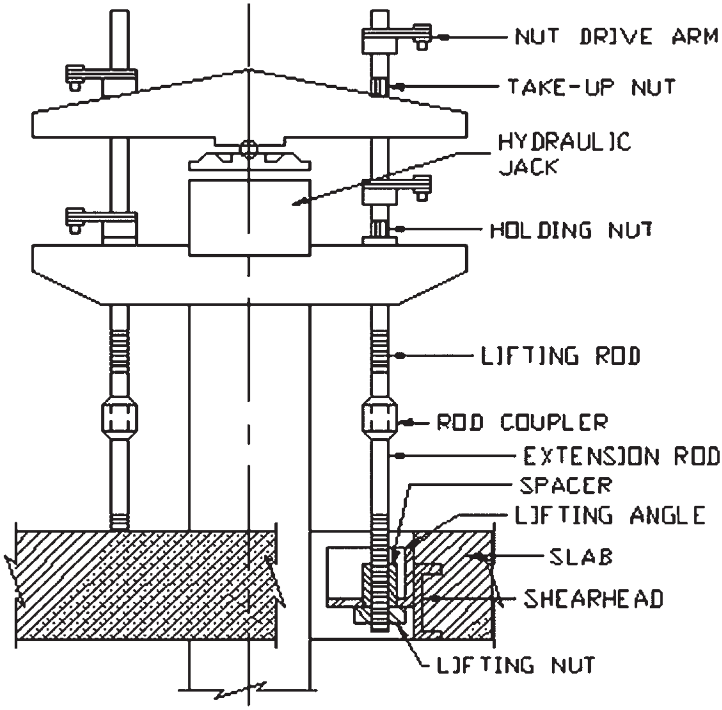

Figure 4-13b. Lifting assembly.
Source: Martin and Delatte (2000).

Shortly before the collapse, the workers lifted the 9th-to-11th floor package to its final position and began tack-welding the steel wedges into place. They used a jack on top of the column E4.8 or E3.8 to slightly adjust the position of the slab, overloading the lifting angles. When the shear heads and lifting angles had lifted the package of three 3.13 MN (320-ton) slabs, they were dangerously close to their maximum capacity, so adding even the smallest of loads could exceed that maximum.

The lifting capacities of the two types of jacks used were too small for the 9.38 MN (960-ton) package. The regular jacks have a maximum load of 869 kN (89 tons), whereas the super jacks have a maximum load of 1.47 MN (150 tons). NBS also tested the shear head and lifting angle and found that the angles tended to twist as the loads approached 781 kN (80 tons), because although the angles had enough strength, they did not have enough stiffness. The force deformed the lifting angle, allowing the jack rod and lifting nut to slip out of the lifting angle and hit the column with 333 kN

(75,000 lb) of force. This load accounted for the loud noise that Kenneth Shepard heard and the indentation found in that column. After this initial slip, the jack rods and lifting nuts in the entire E line progressively slipped, causing the 9th floor slab to collapse, initiating the collapse of the entire building (Korman 1987).

This theory was published before evidence pointed out errors in the alleged facts (Zallen and Peraza 2004). However, the proponents of this theory believe that it is still supported by the available evidence (Culver 2002).

Theory 2—Unstable Wedges

Thornton-Tomasetti Engineers (T-T) concluded that the instability of the wedges at column 3E caused the 12th floor–roof package to fall, initiating the collapse. They disagreed with the NBS investigation, finding that all the wedges supporting the 9th-to-11th-floor package were mounted before the collapse and that that column had no indentations on it. They, however, did find abnormal tack welds on the wedges that supported the 12th floor–roof package, a large deformation on the top edge of the west wedge of this set, and indentations on the underside of the level 9 shear head. The shallowness of the indentations indicated that, although both lifting nuts slipped out, they were not heavily loaded at the time.

Their investigation also found that the shear head gaps on columns 3E and 3.8E (16 mm, 0.628 in.) were much larger than the gaps on the rest of the building (5.92–8.31 mm, 0.233–0.327 in.) and other buildings built with the lift-slab technique (6.35–9.53 mm, 0.250–0.375 in.). In addition to these abnormally large gaps, the shear heads used on these two columns did not have cutouts in their lifting angles to restrict shifting, and they were installed eccentrically. Finally, until a wedge was completely welded into place, it depended on friction to hold it. Normally, friction is sufficient. The large shear head gaps on columns 3E and 3.8E and the presence of hydraulic fluid on these wedges, however, would have demanded an extremely high friction coefficient to hold the wedges in place.

On the day of collapse, the lateral load from the hydraulic jack was exerted on the heavily loaded wedges, causing the west wedge to roll. Then the local adjustments to slab elevations caused the remaining wedge to roll out, initiating the collapse of the 12th floor–roof package and the west tower (Fig. 4-14). Forces transmitted through the pour strips or the horizontal jack, or the impact of the debris from the west tower, triggered the east tower's collapse (Cuoco et al. 1992). Zallen and Peraza (2004, pp. 28–29) refer to this as "theory 3—wedges falling out."

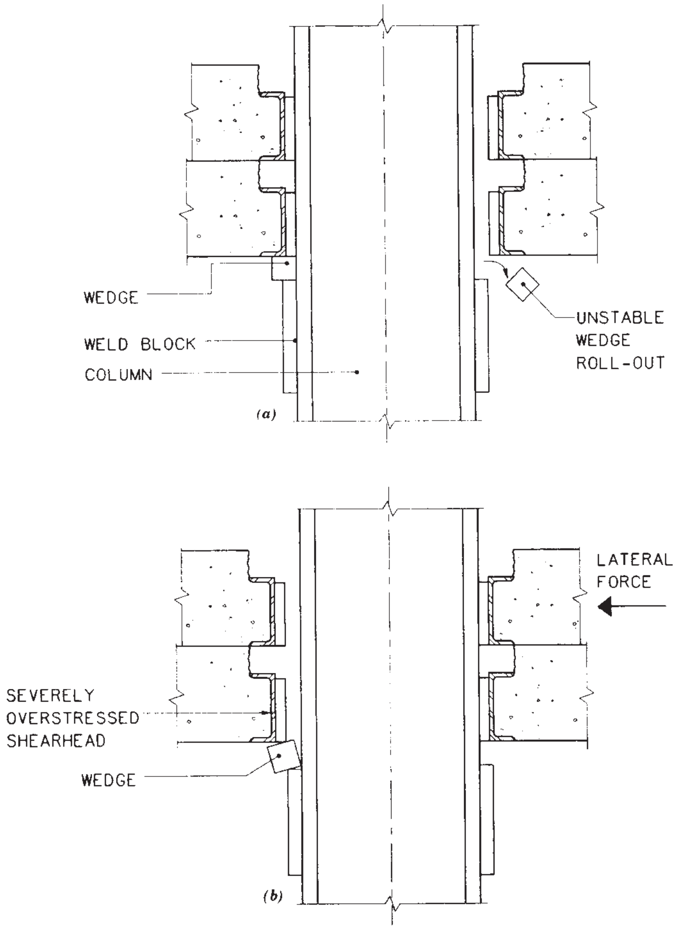

Figure 4-14. Wedges and wedge roll-out mechanism.
Source: Cuoco et al. (1992).

Theory 3—Improper Design of Post-Tensioning Tendons

SSE analyzed the structural behavior of a typical west tower floor slab with respect to the unusual layout of the post-tensioning tendons (Fig. 4-15). The tendons in the east tower followed a typical two-way banded post-tensioning tendon layout. In this layout, the vertical tendons distributed the weight of the slab to the east–west column lines, which in turn distributed

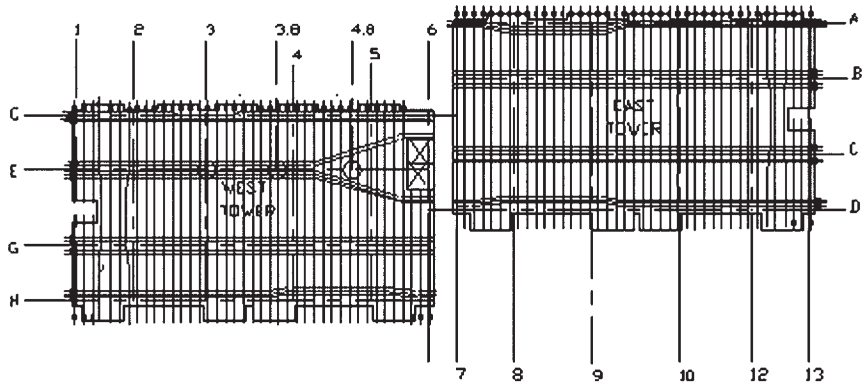

Figure 4-15. Post-tensioning tendon layout.

the weight to the columns. The west tower, however, deviated from this pattern. At column 4.8E, the tendons split in two, both diverging from the column line. In the west tower, the vertical tendons still distributed the slab's weight to the column line. Along line E, however, there are no tendons to carry this weight. This setup violated the American Concrete Institute Building Code (ACI 1983). Kaminetsky points out that the code stipulates "a minimum of two tendons shall be provided in each direction through the critical shear section over columns" (1991, p. 84).

Furthermore, the design details of the post-tensioned floor slabs do not show the location of the shear walls or the openings for the walls at columns 11A, 8A, and 2H. The design did not take these openings into account. Detailed finite-element analysis showed that tensile stresses along column line E, east of column 4.8E, exceeded the cracking strength of the concrete. Therefore, once a crack began, it would immediately spread to column 4.8E. In addition, under ideal lifting conditions, the analysis demonstrated that column 2H would have high compressive and punching shear stresses (Poston et al. 1991). Zallen and Peraza (2004, p. 29) refer to this as "theory 5—improper post-tensioning design."

Theory 4—Poor Weld Details and Welds

OSHA found that the header bar-to-channel welds on one side of the 9th floor shear head at column E3.8 had failed. The use of one-sided square-groove welds for the header bar-to-channel connection was criticized because they were not prequalified joints, according to American Welding Society standards. Because the amount of weld penetration was not known, their strength could not be determined. OSHA hired Neal S. Moreton and

Associates to examine 30 welds around the shear heads at column E3.8 at the 7th, 8th, and 10th floors. They found only 13 of the 30 welds acceptable; the other 17 were substandard. The questionable weld details and substandard welding, coupled with drawings that indicated that the welds would undoubtedly experience forces that they could not resist, all point to weld failure as the trigger of the collapse (McGuire 1992).

Zallen and Peraza (2004, p. 28) refer to this as "theory 2—failure of welds in lifting collar." After weld failure, load would be transferred to the lifting angles, which would then fail in turn.

Theory 5—Global Instability

The FaAA studied the towers' torsional stability and response to lateral loading to understand their collapse. When the concrete slabs were temporarily resting on the wedges, the connection was rotationally stiff, but as soon as the slab was lifted off one of the wedges into its final position, it could rotate freely from the column. Once the wedges were fully welded into their final position, the connection became rigid again. In the absence of lateral loading, the towers were completely stable.

Lateral loading and displacement, however, could cause the slab to lift off one of its wedges, causing the structure to become laterally flexible. The FaAA used 3D computer modeling and nonlinear stability modeling to study this phenomenon. Their investigation and analysis led them to the conclusion that the towers' sensitivity to lateral displacement caused their collapse. Whereas the FaAA acknowledges that another mechanism could have triggered the lateral displacement, they believed that lateral jacking provided sufficient displacement to initiate the collapse (Moncarz et al. 1992). Zallen and Peraza (2004, p. 29) refer to this as "theory 4—instability."

Theory 6—Foundation Failure

In a discussion replying to Cuoco et al.'s 1992 paper, Rendon-Herrero suggests that "a closer look warrants reconsideration of the role played by the foundation in the collapse" (Rendon-Herrero 1994). He notes that the NBS report found disintegrated rock, bedrock, and fill materials of varying quality, with some questions as to whether testing of in-place density was performed and as to the rationale for the assumption of the allowable bearing pressure. He concludes that

> The writer feels that descriptions like "mica," "micaceous schist," "highly fractured," "cracks," "disintegrated rock," "fill," "compaction with backhoe," "highly weathered," "thinly laminated," and "very steep dip (nearly vertical)" are red flags that indicate the need for

> caution and special attention in the design of a foundation. Punching or local shear is likely when subgrade conditions include loose granular soils (i.e., inadequate compaction); micaceous soils; micaceous schists; and highly fractured, steeply dipping bedrock. (Rendon-Herrero 1994)

Legal Repercussions

All of these theories are plausible, but what triggered the collapse? The answer may never be known. A two-judge panel mediated a universal settlement among 100 parties, closing the L'Ambiance Plaza case. Twenty or more separate parties were found guilty of "widespread negligence, carelessness, sloppy practices, and complacency." They all contributed, in varying amounts, to the $41 million settlement fund. Those injured and the families of those killed in the collapse received $30 million. Another $7.6 million was set aside to pay for all of the claims and counterclaims among the designers and contractors of L'Ambiance Plaza.

Although this settlement kept hundreds of cases out of court and provided rapid closure to a colossal collapse, it also ended all investigations prematurely, leaving the cause of collapse undetermined (Korman 1988). Fortunately, many of the investigators subsequently published their findings (Feld and Carper 1997).

Technical Aspects

Although buildings constructed by the lift-slab method are stable once they are completed, if great care is not taken during construction they can be dangerous. Feld and Carper (1997) reviewed a number of previous lift-slab construction failures and near-failures. The following measures can be taken to ensure lateral stability and safety during construction:

- During all stages of construction, temporary lateral bracing should be provided, unless the lateral stability of the structure is provided through another mechanism.
- Concrete punching shear resistance and connection redundancies should be provided in the structure (Kaminetzky 1991).
- Sway bracing (cables that keep the stack of floors from shifting sideways) should be used. This bracing was required but not used in L'Ambiance Plaza (Levy and Salvadori 1992).

Because of the terms of the settlement, many of the technical lessons that could have been learned from this incident may have been lost.

Professional and Procedural Aspects

The L'Ambiance Plaza collapse highlighted several procedural deficiencies. Responsibility for design was fragmented among so many subcontractors that several design deficiencies went undetected. If the engineer of record had taken responsibility for the overall design of the building or a second engineer had reviewed the design plans, these defects probably would have been detected (Heger 1991). Also, standardized step-by-step procedures for lift-slab construction should be established to ensure the safety of the construction workers. A licensed professional engineer should be present during construction to ensure that these guidelines are followed (Kaminetzky 1991).

According to Zallen and Peraza, three structural engineers should be involved in the design and construction of a lift-slab building. These are the structural engineer of record, the lift-slab engineer, and the post-tensioning engineer. The structural engineer of record is responsible for the integrity of the building in its completed state. The lift-slab engineer, hired by the lift-slab contractor, designs the lift-slab process, including structural stability during lifting operations. The post-tensioning engineer details the tendons and related details and must coordinate carefully with the lift-slab engineer. All three engineers must coordinate their work carefully (Zallen and Peraza, 2004, pp. 62–63).

Ethical Aspects

Although the L'Ambiance Plaza building was designed to be safe once it was completed, during construction it did not have an adequate level of stability. This situation is all too common in the construction industry today (Heger 1991). Canon 1 of the American Society of Civil Engineers (ASCE) Code of Ethics states, "Engineers shall hold paramount the safety, health and welfare of the public and shall strive to comply with the principles of sustainable development in the performance of their professional duties" (ASCE 2006). This safety includes the safety of construction workers. Building regulations do not sufficiently consider structural safety during construction; they should be changed to require a high standard of safety during construction, as well as after a building's completion. In the absence of such regulations, however, an ethical engineer must always consider the safety of the workers (Heger 1991). The ASCE Code of Ethics is provided in Appendix B.

The Human Factor

Many of the organizations involved in the L'Ambiance Plaza project went out of business. One exception was the structural engineer, James

O'Kon. In October 1991, he described his experiences to his colleagues at an annual ASCE conference.

> A federal judge has just told James O'Kon that he wanted $2 million from him—$1 million in insurance money and $1 million from O'Kon's personal funds. . . . The judge, Robert C. Zampano, was trying to collect enough money to settle all the lawsuits from the Bridgeport collapse and to provide victims and their families with enough money to live comfortably and educate the victims' children. But O'Kon couldn't see why he should have to help make the families "wealthy." He had been exonerated of wrongdoing and, in his opinion, the workers themselves caused the accident. He also told the judge that his daughter had recently been paralyzed in an accident and he had spent hundreds of thousands of dollars in medical expenses. Zampano eased his demands and asked only for the insurance money, thus saving O'Kon from financial devastation. (Houston 1991)

O'Kon believed that workers' errors in operating the lifting jack triggered the collapse. He immediately contacted his clients to assure them that the accident was not his fault and spoke with the media as little as possible. He had, unfortunately, let his Connecticut license lapse, and had to fight for two years to regain it.

In the unpublished remarks presented at the ASCE meeting, more than four years after the event, O'Kon detailed the pressure the judge put on him and others. He was told that if he didn't pay the requested amount, he would be excluded from the settlement group and financially destroyed in litigation. The hot dog man who provided coffee and sandwiches for the laborers paid $75,000, and the drywall metal framing installation company and drywall installer paid $450,000 and $150,000, respectively, although their work had not been scheduled to start for almost six months. The general contractor and developer paid roughly the same amounts as O'Kon, and both went out of business. The supervising architect paid only $7,500 because he did not have insurance.

Other Lift-Slab Cases

There were other problems in the early days of lift-slab construction, just as there often are with innovative construction technologies. The 1962 system patented by Stubbs used a system with grooves spaced 150 mm (6 in.) apart, with jacks that could not be removed if they jammed between strokes. A Canadian wedge system building under construction in Marion, Indiana, collapsed in 1962. The Canadian wedge connection relied on frictional resistance between the column and wedges. Other buildings failed in sidesway mode because of global instability. These buildings included the Junipero Serra High School Roof in San Mateo, California, in 1954 and the Pigeonhole Parking Garage in Cleveland, Ohio, in 1956. The Pigeonhole Parking Garage swayed 2.1 m (7 ft) in winds gusting from 56 to 97 km/h (35 to 60 mi/h), but the contractor was able to bring it back to plumb (Zallen and Peraza 2004, pp. 22–25). This near-collapse is discussed in the next section. Unlike L'Ambiance Plaza, none of these failures led to a loss of life. These incidents highlighted possible problems during jacking operations, including uneven lifting of slabs, as well as the importance of overall stability.

Conclusions

The L'Ambiance Plaza collapse killed 28 workers and had serious ramifications for all the people involved with the project, as well as for the civil engineering profession as a whole. All of the theories discussed above are plausible, but it seems unlikely that the triggering mechanism of the collapse can ever be determined. This failure severely reduced the use of the lift-slab system and almost eliminated it from use (Moncarz and Taylor 2000, p. 46). The ASCE Technical Council on Forensic Engineering (Task Committee on Lift-Slab Construction) published *Engineering Considerations for Lift-Slab Construction* to benefit future designers and builders of lift-slab projects (Zallen and Peraza 2004).

Essential Reading

Zallen and Peraza (2004) provide an important review of the technical aspects of lift-slab construction. An account is provided by Levy and Salvadori (1992). This case study is featured on the History Channel's Modern Marvels *Engineering Disasters 4* videotape and DVD.

Cleveland Lift-Slab Parking Garage

Although the final cause of the L'Ambiance Plaza collapse has never been determined, the reason of the 1956 near-collapse of the Pigeonhole Parking Garage in Cleveland, Ohio, appears obvious. This was also a lift slab structure.

The building consisted of eight-story twin towers, 28 × 6.4 m (91 × 21 ft) each in plan. The slabs were 200 mm (8 in.) thick and weighed 800 kN