

INVESTIGATION OF THE OCTOBER 24, 2001 FATAL COLLAPSE OF TWO SCAFFOLD TOWERS AT 215 PARK AVENUE SOUTH IN NEW YORK, NY

U. S. Department of Labor
Occupational Safety and Health Administration

February 2003



THIS PHOTOGRAPH WAS TAKEN AT THE BEGINNING OF THE RESCUE OPERATION WITHIN HOURS OF THE COLLAPSE (REPRODUCED FROM NEW YORK DAILY NEWS, THURSDAY, OCTOBER 25, 2001).

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COLLAPSE OF TWO SCAFFOLD TOWERS AT 215 PARK
AVENUE SOUTH IN NEW YORK, NY**

Occupational Safety and Health Administration
Directorate of Construction
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This report was prepared by
Scott Jin, Ph.D., P.E.

1. EXECUTIVE SUMMARY

On October 24, 2001, at approximately 4:00 PM, two 142 ft high scaffold towers collapsed, killing five workers and injuring ten others. The towers were erected to provide a working platform for the facade renovation of a 20-story steel framed masonry building. The 1914 era building is located at 215 Park Avenue South in New York City. On the day of the incident, the face bricks and steel window lintels had already been removed from the 6th to the 14th floors and the cement plaster work had just begun at the 14th floor of the west wall. At the time of the incident, two workers were manually hoisting a cement bag to the top of the scaffold tower.

Personnel from Manhattan Area Office of Occupational Safety and Health Administration (OSHA) arrived at the scene within hours of the incident. The Directorate of Construction, OSHA, Washington, DC, was requested to provide assistance in technical assessment of the collapse and in determining the cause of the incident.

The OSHA investigation began soon after the incident and included interviewing witnesses, observing failures, inspecting recovered steel frames and supporting members, taking photographs and related field measurements, and performing an engineering analysis. Based on the findings of the investigation, it is concluded that:

1. The two scaffold towers were not designed to comply with OSHA Standard 1926.451(a)(1). The towers were intended to support the load of two levels of bricks removed during the façade renovation, equivalent to a live load of 32 psf for each level. By applying four times the maximum intended loads, the following main structural members would fail due to overstressing beyond their ultimate capacities.
 1. Thirteen of eighteen scaffold legs (posts) for both towers.
 2. Three steel beams (Dunnage Nos. 1, 2, and 3 from north) supporting the northern portion of the towers.
 3. The south aluminum beam supporting Legs 17 and 18.
 4. The south horizontal member at the top of the lower offset scaffold.
 5. The 12-foot long pegleg supporting Leg 15.

Thus, Section 1926.451(a)(1) was violated.

2. The two scaffold towers were approximately 142 ft in height. Due to the different base elevations within the footprint of the towers, three of the eighteen legs were extended two levels below the main base elevation. Thus, the total height of a portion of the scaffold towers was 155 ft. In either case, OSHA Standard 1926.452(c)(6) was violated because the towers were not designed by a professional engineer (PE).

OSHA Standard 1926.452(c)(6) requires that all scaffolds over 125 ft be designed by a PE.

3. In the west scaffold tower, the southernmost frame was extended two levels below the main base elevation. The two-level frame was not properly braced in accordance with the manufacturer's recommendation. In the east tower, a 12 ft long structural tube was used to extend the tower leg (pegleg) to provide bottom support for the west end of the fourth frame from the north. This tube was neither a manufacturer's recommended main scaffold member nor was it designed by a PE.
4. Either the overstressing of the three dunnage beams or the pegleg caused the failure. It is difficult to ascertain which of these two factors had a predominant role.

2. CONDUCT OF INVESTIGATION

The following is a chronological list of major events during the investigation.

On October 24, 2001, with hours of the incident, personnel from the Manhattan Area Office of the Occupational Safety and Health Administration (OSHA) arrived at the scene to start the investigation by interviewing witnesses and taking photographs.

On October 26, 2001, at the request of OSHA Region II, two engineers from OSHA's Directorate of Construction (DOC), in Washington, DC, accompanied by two compliance officers from the Area Office, visited the incident site to inspect the collapsed scaffold towers and take photographs and field measurements. OSHA personnel also recorded the condition of the façade, the windowsills, and the remaining lateral bracing points. The Area Office reviewed the project organization, the responsibility of contractors and subcontractors, and the sequence of events leading to the incident.

On November 2, 2001, DOC received the first package of documents from the Area Office, which included the following:

- A two-page handwritten note, prepared by the scaffold erector, Tri State Scaffold & Equipment Supplies Corporation (TSS). The first page of this note was a simplified sketch of the proposed dimensions of the scaffold tower. The second page was a list of scaffold parts and cost estimates. According to the owner of TSS, the scaffold towers were not designed by a professional engineer (PE). The two-page note was the only document used for erection purposes.
- Two signed interview statements. The first statement was from the owner of TSS; the second from one of his employees. These two documents assert that four aluminum I-beams were placed on top of dunnages of the air conditioning units at the second floor level to serve as the base sills for 12 legs of the scaffold towers.
- Ten black and white digital photo printouts. Five photos depict the arrangement of the scaffold towers before the collapse; the other five indicate the condition of the towers after the collapse.

On November 14, 2001, DOC received a second package of documents from the Area Office, which included the following:

- Preliminary plan and profile of the base support system of the scaffold towers developed by the principal compliance officer of the project. Both sketches were prepared based on the field measurements.
- Bil-Jax catalog 9S6, capacities of Bil-Jax size 7 frame, and general plan and elevations of the 20-story steel framed masonry building. These documents were furnished by TSS. Please note that the plan and elevations of the existing building indicated the limits of face bricks to be removed.

- Contract agreement between S. L. Green Realty Corporation (SLG) and Nesa Roofing and Restoration, Inc. (NRR) and contract agreement between NRR and TSS. Both documents were provided by NRR. Please note that SLG was the property's leasing and managing agent, while NRR was the general contractor.

On November 16, 2001, an engineer from DOC, accompanied by two compliance officers from the Area Office, re-visited the incident site to take additional photographs and detailed measurements of the collapsed scaffold members, and conduct a meeting with the owner of the scaffold erector (TSS). The following are the highlights:

- TSS stated that the two scaffold towers were erected in the south courtyard. The west tower had four frames with three bays of 8 ft, 8 ft, and 7 ft in length from north to south. The east tower had five frames with four bays of 8 ft, 8 ft, 7 ft, and 8 ft in length. Please note that the last (south) bay of the west tower was a stair tower that was manufactured with a fixed length of 7 ft, explaining why all bays were not 8 ft in length. The identification of scaffold frames and legs of the two scaffold towers are presented in Figure A-1 of Appendix A.
- TSS stated that both east and west towers were tied together by a lateral bracing system installed at every fourth level of the tower inside the courtyard. As a result, a total of six levels of bracings were placed. Approximately 8 to 10 pressure pads per bracing level were used. In addition, regular horizontal ties were installed from the scaffold towers to the existing windowsills.
- TSS stated that the existing handrails within the footprint of the towers were not used as part of the vertical support system or as the lateral bracing system. As a result, handrails at the mezzanine level were used only to fix the position of the lower frames and a 12 ft long individual leg (pegleg) during the installation. These two structural members were eventually braced near the top by horizontal bracers to the north.
- TSS stated that the erection of the scaffold towers started on Saturday, October 6, 2001. The erection was completed and released for use on Tuesday, October 16, 2001. Please note that this statement implies that the scaffold towers were fully planked.

During the November 16 trip, DOC received 50 color photographs and 21 black and white digital photo printouts taken by TSS prior to the collapse. In addition, the evaluation of the actual load at the time of the collapse was also discussed with the Area Office.

On December 11, 2001, after the face bricks and debris were partially removed from the roof deck at the second floor level, the engineer from DOC, accompanied by the same personnel from the Area Office, re-visited the incident site, to measure the actual size and length of the newly exposed scaffold supporting members, inspect the failed condition of

the east end of three dunnages, and weigh the scaffold planks. During this trip, DOC received a copy of the installation manual for the disposal chutes (Chutes International). On December 26, 2002, the Area Office provided the DOC with a copy of 5-page letter size documents of the 1991 plan, section, and detail of the design condition of the existing dunnages in the south courtyard.

On February 19, 2002, after all key structural members were recovered from the roof deck area, the engineer from DOC, accompanied by the OSHA principal compliance officer, visited the incident site to examine the deformed conditions of recovered members and take associated measurements. During the same trip, DOC presented the preliminary findings of the investigation to the Area Office and regional solicitors.

On February 28, 2002, DOC received the following information from the Area Office:

- The initial weight of bricks and debris of 16.5 tons, recovered from the roof deck at the second floor level, as reported by the City Consultant, Wiss, Janney, Elstner Associates, Inc. (WJE).
- The dimension of the dunnage holes after the collapse as measured by the OSHA Area Office.
- Eleven black and white copies of digital photos of recovered structural members, taken by the OSHA Area Office.
- The proposed testing schedule prepared by WJE.

On February 7, 2003, the Manhattan Area Office provided the DOC with the following information:

- The final weight of bricks and debris of 20.6 tons on the scaffold towers at the time of the collapse as estimated by WJE.
- The laboratory test results for the investigation as performed by Lucius Pitkin, Inc.

Throughout the course of the investigation, DOC worked with personnel from the OSHA Manhattan Area Office. The principal compliance officer, Marc Vargas, and the compliance officer, Robert B. Stewart, made significant contributions to this investigation. Dinesh Shah, structural engineer of the Office of Engineering Services, DOC confirmed the calculations of the draft report.

3. DESCRIPTION OF THE PROJECT, THE INCIDENT, AND THE COLLAPSE

3.1 Description of the Project

The project involved the renovation of the façade at the south courtyard of a 20 story steel frame masonry building. The building, constructed in 1914, is located at 215 Park Avenue South in Manhattan, New York City (See the insert project location plan in Figure 3-1). The renovation work included restoration of existing steel shelf angles (window lintels) and replacement of face bricks at the south interior courtyard. The proposed work limits were from the 2nd to the 14th floors at all three sides of the courtyard, as shown in Figures 3-1 through 3-3. A project organization chart is included in Figure 3-4.

To provide an adequate work platform for the masonry contractor, the general contractor retained a scaffold erector to build two scaffold towers in the south courtyard. The existing condition at the base of the courtyard prior to the scaffold erection is presented in Figures 3-5 and 3-6. This condition was the result of installation of an HVAC system in 1991. As indicated, roof decks at the second floor and mezzanine levels covered the entire courtyard, but were not capable to support any scaffold leg loads. There were three steel beams (dunnages) just above the roof deck at the second floor level to support the four existing air-cooled-condensing-units (ACCU). These beams were anchored to the building wall (Figure 3-6). Thus, the scaffold erector chose to use these beams to support the majority of the scaffold loads without performing any structural analysis.

Twelve of the eighteen legs of both towers were supported on the three existing steel beams (Figures 3-7 and 3-8). The next three legs were supported on top of the parapet wall at the mezzanine level. The last three legs at the southeast corner were primarily supported on an offset scaffold, built from the first floor level (Figures 3-9 and 3-10). Pressure jacks were also used along with the regular horizontal ties for lateral bracings and for connections between the two scaffold towers (Figures 3-11 and 3-12). Both towers were fully planked and completed on Tuesday, October 16, 2001, and released to the general contractor. The masonry contractor began work shortly thereafter on the two towers. The scaffold erector visited the scaffold towers on October 17, 20, and 24, 2001. The October 20 visit was to repair the disposal chute. In each of these visits, the scaffold erector had to re-tighten about 10 to 15 pressure jacks, which had loosened as a result of brick removal by the masonry contractor during the demolition operation.

3.2 Description of the Incident

On the day of the incident, October 24, 2001, face bricks and window lintels were removed from the 14th through the 6th floors on all three sides of the façade. New cement plaster work had begun from 14th through 11th floors on the west wall. Based on witness accounts, demolition bricks had accumulated in piles up to four feet high on the upper levels of both towers. However, no noticeable settlements or deflections were reported prior to the collapse.

Immediately prior to the incident, around 4:00 PM, two workers were manually hoisting a 94-pound cement bag from the first floor with a well wheel, supported on the hoist arm at the top level of the east tower and connected to the west leg of the southernmost frame. It was reported that the hoisting operation caused vibrations and sways of the towers. With no warning, both scaffold towers collapsed with a loud crashing sound. The building trembled as both towers fell.

As a result of the collapse, two 12-story high scaffold towers were reduced to a 3-story high pile of twisted metal and splintered wooden planks that filled the south courtyard. Many of the victims were buried in this pile of rubble. The incident caused the death of five workers and injured ten others.

3.3 Description of the Collapse

At the collapse, many workers were buried in the pile of rubble. A feverish rescue operation began involving more than 250 policemen and firefighters. During the rescue operation, most of the scaffold frames were saw cut and removed along with scaffold planks from the courtyard through the window openings into rooms at the 3rd and 2nd floors of the building. These materials were transported directly to a disposal site on Staten Island, New York City.

Based on the photograph on the cover of this report, the green colored "Scaff-Trac" hoist frame and the white colored electrical hoist (shown in Figure 3-12), landed in the southwest corner of the courtyard. Thus, the scaffold towers fell in a southwest direction. This observation is consistent with the witness accounts.

The collapsed conditions of the three steel beams supporting twelve scaffold legs are shown in Figures 3-13 through 3-18. As indicated, the west ends of all three beams were still attached to the building wall, while their east ends were either twisted or separated. Thus, the east end of the beams failed first, followed by the mid portion of the beams landing on the roof deck. The east end of the south beam was totally separated from the building wall and landed on the roof deck. However, the east ends of the remaining two beams were twisted but still attached to the building wall. Thus, it is believed that the east end of the south beam failed first.

The failed condition of the pegleg (which supported scaffold leg L15) from top to bottom is presented in Figures 3-19 through 3-22. As indicated, the pegleg showed a typical buckling failure under the applied load (Figure 3-21). Bending at its upper bracing connection (Figure 3-19), but remaining straight at its lower connection to the handrail (Figure 3-20). Thus, the handrail deflected with the pegleg, providing little resistance against buckling.

The deflected conditions of the top horizontal members at the upper frames of the offset scaffold, built from the first floor level, are shown in Figure 3-23. As indicated, the north top horizontal member adjacent to the pegleg experienced more deflections than the south

top horizontal member, although the south carried more loads than the north prior to the collapse. In addition, as a result of the collapse, the second level handrails close to the pegleg were separated and removed from the roof wall, while those further away remained intact, as shown in Figure 3-24. The rotational (to the south) failure of leg L17 above the south top horizontal member of the offset scaffold is shown in Figure 3-25. The improper bracing condition of the bottom frame, which supported scaffold legs L7 and L8 from the parapet wall at the mezzanine level is presented in Figure 3-26.

The condition of the recovered scaffold frames, lateral bracings, and bottom support members are presented in Figures 3-27 through 3-30. As indicated, among the three W8X13 beams (dunnages), the south beam had the greatest deflections consistent with the above description. The other deformed members included one I8X7.02 aluminum beam, the pegleg, all lateral braces, and all scaffold frames. However, four W4x13 steel beams connecting ACCUs to the three dunnages and three recovered I8X6.35 aluminum sills remained intact. This implies that the three W8X13 beams, four upper W4X13 beams, and four upper I8X6.35 aluminum sills in the perpendicular direction (Figures 3-5 and A-8) failed as a unit, with the failure starting from the south side. The two S6X12.5 steel beams also remained intact. Perhaps the applied loads on the two beams were within their yield limit.

4. ANALYSIS AND DISCUSSION

The collapsed scaffold towers in the south courtyard were not designed by a PE and no construction drawings were prepared. The adequacy of the towers was evaluated based on their size and height, and the applied loads. The plan dimensions of both towers were ascertained based on the scaffold erector's statements and photographs taken during the erection, then verified with the manufacturer's catalog and OSHA's field measurements taken after the collapse (Figure A-1 of Appendix A). Scaffold frames and legs of the towers are identified in this figure. The configuration of the 22-level high scaffold towers was shown in Figure A-5 using information derived from a series of photographs (Figures A-2 through A-4). For clarity, the tower is divided in two parts, one above the main base plate level and the other below. The main base plate level is the top elevation of the four aluminum sills. The referenced figures and sections with a letter "A" are contained in Appendix A.

4.1 Scaffold Tower Frames above the Main Base Plate Level

The first step was to determine the leg loads of the scaffold based on weight (dead load of scaffold material plus planks). These are calculated in Section A-2, tabulated in Table A-1, and compared against allowable loads. The allowable leg load for a 6 ft 4 in high frame is 1.89 kips (Figure A-6). It was determined that the actual loads in all eighteen legs exceeded allowable leg loads. The weight of planks contributed 64% to the leg loads. However, the actual loads were still considered within the ultimate capacity of 7.56 kips, but without the required factor of safety of four. The ultimate capacity of the leg was calculated by multiplying the allowable load by four.

Next, the leg loads were computed due to the dead load as discussed above plus four times the additional loads of bricks to be removed from the building façade. The weight of bricks was established based on the written statements from the scaffold erector, i.e., "They (masonry contractor) could load no more than two levels of the scaffold." and "Two levels of brick are enough for the job." One purpose of the investigation was to determine whether the scaffold and support members could resist four times the intended load. Therefore, the weight of the bricks was multiplied by four to determine compliance with the OSHA standard. The leg loads, calculated in Section A-4 and tabulated in Table A-2, exceeded the allowable loads in all eighteen legs. In 13 of 18 legs, the loads exceeded even the ultimate capacity of the legs. Thus, Section 1926.451(a)(1) was violated.

Third, the leg loads were computed based on the weight of the scaffold and the actual brick loads placed on the scaffold before the incident. The actual load was provided by the Manhattan area office, based on the weight of bricks recovered after the incident. The actual load was found to be approximately 1.7 times the maximum load the erector intended to place. The leg loads in all eighteen legs, calculated in Section A-4 and tabulated in Table A-3, exceeded the allowable load of 1.89 kips. Six of the eighteen legs were overloaded about 14% beyond their ultimate capacities of 7.56 kips.

4.2 Scaffold Tower Support Members Below the Main Base Plate Level

The scaffold towers were supported as described in Section A-5 and presented in Figures A-8 and A-9. The structural adequacy of the supporting members is examined in Sections A-5-1 through A-5-7 based on their ultimate capacities. Load factors and capacity reduction factors were taken as one. The loads over the support members were computed in the same order as above in Section 4.1.

First, loads due to the weight of the scaffold and the planks were calculated. It was determined that three W8X13 dunnage beams and the 12-foot long pegleg were stressed beyond their ultimate capacities. These structural members are discussed separately in the subsequent paragraph.

Second, the loads based on the weight of the scaffold and four times the weight of two levels of bricks were computed. The following support members were found to be stressed beyond their ultimate capacities:

- Three W8X13 dunnage beams supporting the northern portion of the towers.
- South I8X7.02 aluminum beam supporting the scaffold legs L17 and L18.
- South horizontal member at the top of the lower offset scaffold.
- The 12-foot long pegleg supporting scaffold leg L15.

Thus, Section 1926.451(a)(1) was violated.

Third, the loads of the weight of the scaffold plus the actual load of the bricks (earlier determined to be 1.7 times the load of the two levels of bricks) were calculated. It was again determined that three W8X13 dunnage beams and the 12-foot long pegleg were stressed beyond their ultimate capacities.

Three W8X13 Dunnage Beams

Of all factors, the unbraced length of compressive flange of the dunnage beam had the greatest impact on the load carrying capacity. Crossbeams welded to the dunnage beam provided an unknown degree of lateral and translational restraint, which complicates the determination of the exact amount of unbraced length. The unbraced length might be the full length of the beam or the distance from one end of the beam to the crossbeam. In either case, however, under the actual load placed at the time of the collapse, the beam was stressed beyond its ultimate capacity and failure was inevitable. The beam was overstressed by more than 400% under the full unbraced length and by 140% under the reduced unbraced length.

The 12-foot Long Pegleg

The maximum load that the 12-foot long pegleg could support, without buckling, was determined to be 3.74 kips. However, the loads placed on the pegleg due to (1) the weight of the scaffold (dead load), (2) four times the weight of two levels of bricks plus the dead load, and (3) the actual load of the bricks at the time of collapse plus the dead load were 5.37, 10.17 and 7.45 kips, respectively. As can be seen, the pegleg was on the verge of collapse under the weight of the scaffold. The pegleg was a critical member of the framing, based on its long unbraced length. The direction of the collapse combined with the extensive damage sustained by the structural members adjacent to the pegleg indicate that the tower collapse could have been triggered by the failure of the pegleg, though it is difficult to say which of the two potential causal factors, i.e., the three dunnage beams or the pegleg, performed the predominant role.

5. CONCLUSIONS

The following findings are based on the investigation:

1. The two scaffold towers were not designed to comply with OSHA Standard 1926.451(a)(1). The towers were intended to support the load of two levels of brick removed during the façade renovation, equivalent to a live load of 32 psf for each level. By applying four times the maximum intended load, the following main structural members would fail due to overstressing beyond their ultimate capacities.

- Thirteen of eighteen scaffold legs (posts) for both towers.
- Three steel beams (Dunnage Nos. 1, 2, and 3 from north) supporting the northern portion of the towers.
- The south aluminum beam supporting Legs 17 and 18.
- The south horizontal member at the top of the lower offset scaffold.
- The 12-foot long pegleg supporting Leg 15.

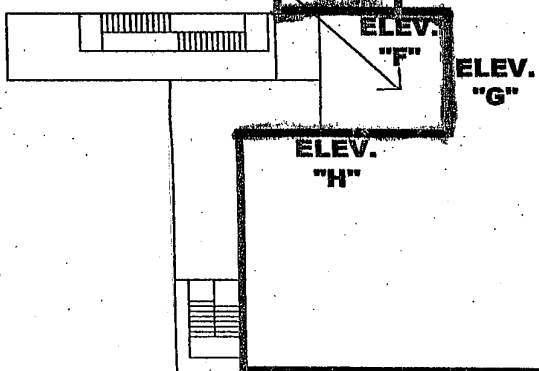
Thus, Section 1926.451(a)(1) was violated.

2. The two scaffold towers were approximately 142 ft in height. Due to the different base elevations within the footprint of the towers, three of the eighteen legs were extended two levels below the main base elevation. Thus, the total height of a portion of the scaffold towers was 155 ft. In either case, OSHA Standard 1926.452(c)(6) was violated because the towers were not designed by a professional engineer (PE). OSHA Standard 1926.452(c)(6) requires that all scaffolds over 125 ft be designed by a PE.
3. In the west scaffold tower, the southernmost frame was extended two levels below the main base elevation. The two-level frame was not properly braced in accordance with the manufacturer's recommendation. In the east tower, a 12 ft long structural tube was used to extend the tower leg (pegleg) to provide bottom support for the west end of the fourth frame from the north. This tube was neither a manufacturer's recommended main scaffold member nor was it designed by a PE.
4. Either the overstressing of the three dunnage beams or the pegleg caused the failure. It is difficult to ascertain which of these two factors had a predominant role.

6. REFERENCES

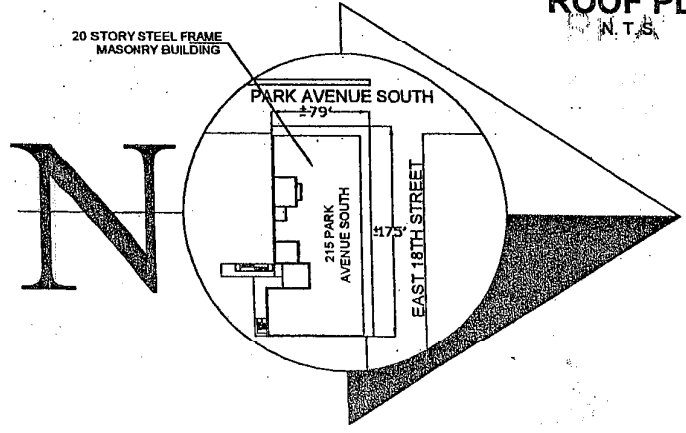
1. Manual of Steel Construction, Allowable Stress Design, American Institute of Steel Construction, Ninth Edition, 1989.
2. Load and Resistance Factor Design, Volume I, American Institute of Steel Construction, Second Edition, 1994.
3. Bil-Jax Catalog 9S6, Bil-Jax, Inc., Archbold, Ohio, 1996.
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7. 215 Park Avenue South, New York, New York, Investigation of the Collapse of the Scaffold System, WJE Engineers & Architects, PC, October 1, 2003.
8. Steel Structures, Design and Behavior, Charles G. Salmon and John E. Johnson, Harper & Row, Publishers, 1971.

**WORK AREA
SOUTH COURTYARD
ELEVATION F, G AND H**



EAST 18TH STREET

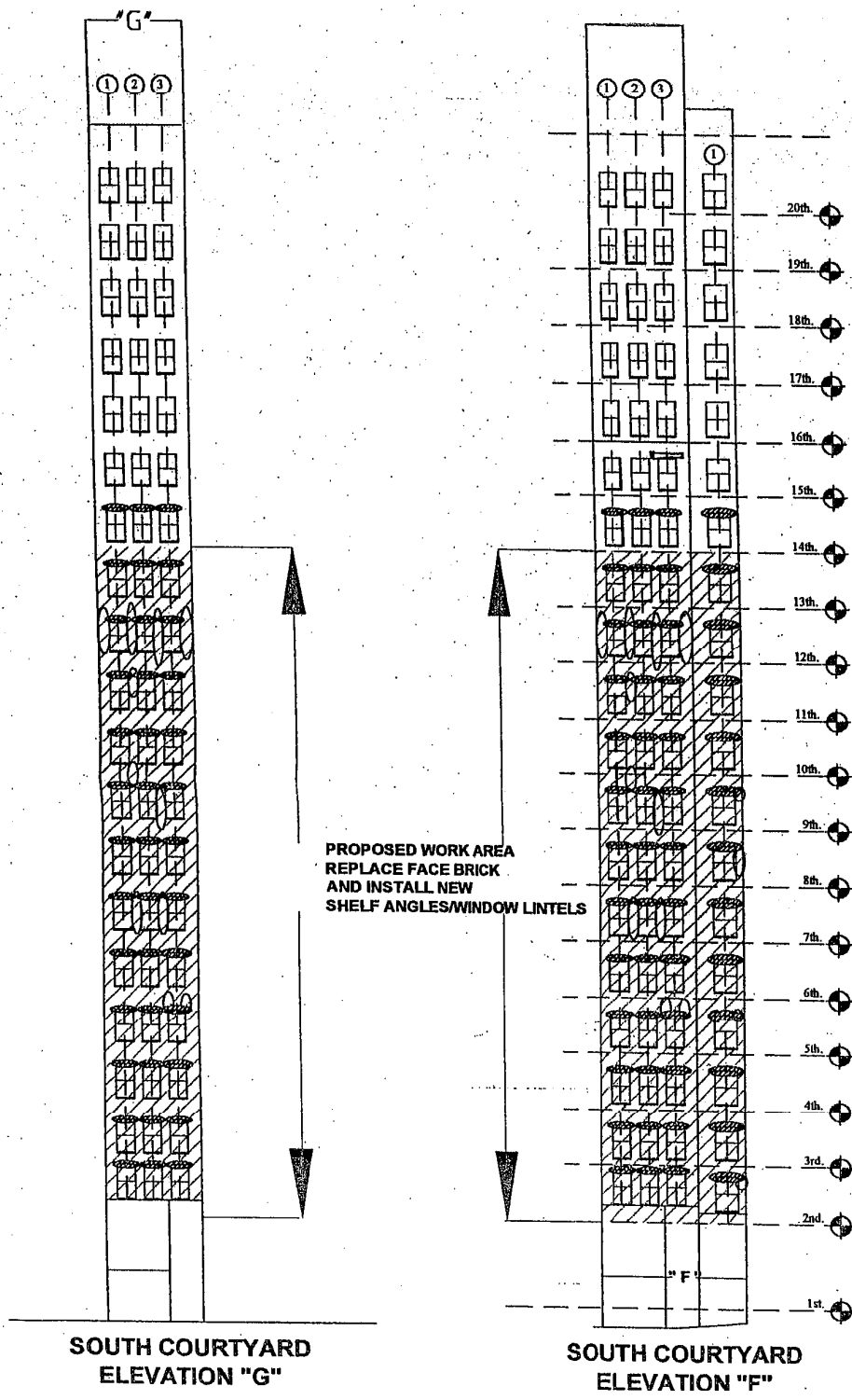
**ROOF PLAN
N.T.S.**



**PLOT PLAN
N.T.S.**

MICHAEL K. KOEHLER P.C. 617 55 STREET BROOKLYN, N.Y. 11220		
FACE BRICK AND SHELF ANGLE/WINDOW LINTEL REPLACEMENT/LINTEL RESTORATION		
215 PARK AVENUE SOUTH		
DRAWN BY: S.A.	CHECKED BY: M.K.K.	
DATE: 06-01-01	SCALE: ---N.T.S.	SH: 1 OF 4

Figure 3-1. Plan View of the Proposed Work Limits (From General Contractor).



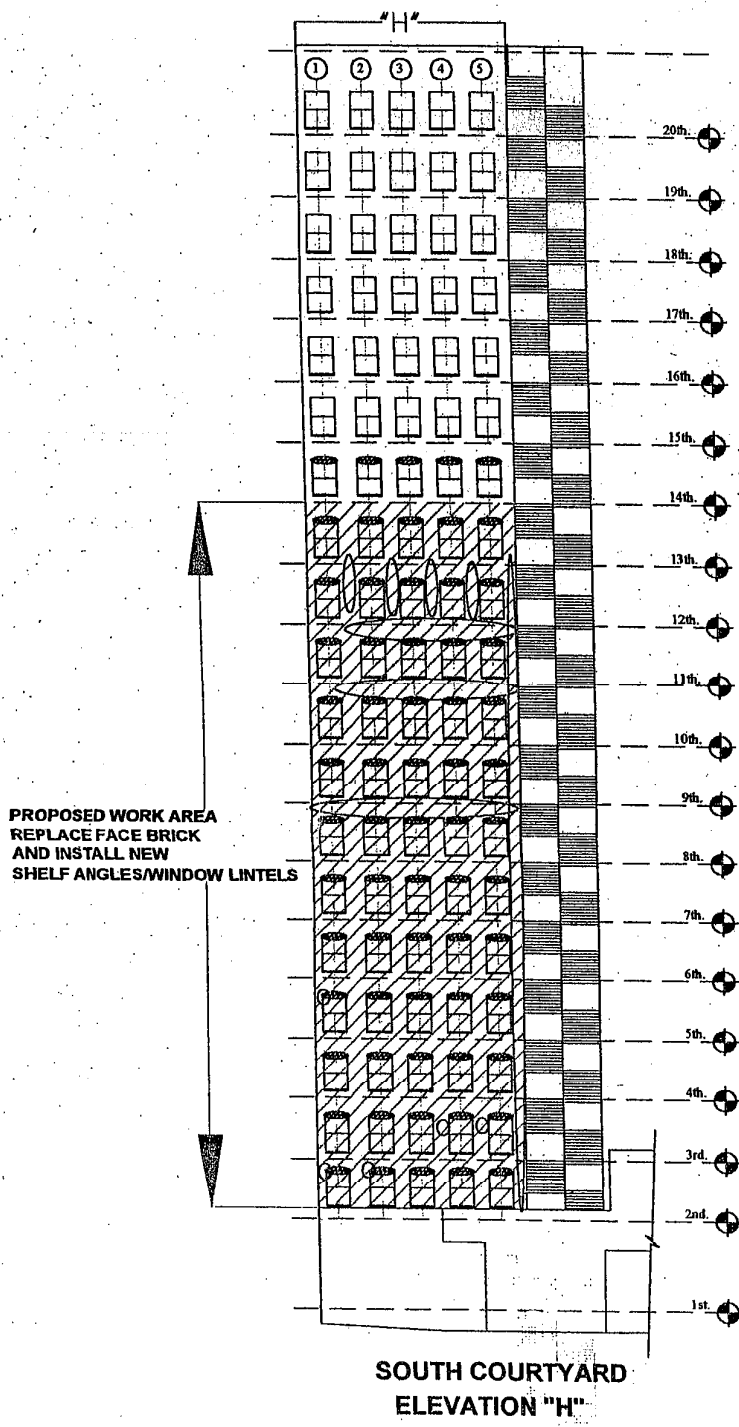
SOUTH COURTYARD ELEVATION "G"

SOUTH COURTYARD ELEVATION "F"

COURTYARD FACADE
N. T. S.

MICHAEL K. KOEHLER F.C.		
<small>617 53 STREET, BROOKLYN, N.Y. 11220</small>		
<small>FACE BRICK AND SHELF ANGLE/WINDOW LINTEL REPLACEMENT/LINTEL RESTORATION</small>		
<small>215 PARK AVENUE SOUTH</small>		
<small>DRAWN BY: S.A.</small>	<small>CHECKED BY: M.K.K.</small>	
<small>DATE: 06-01-01</small>	<small>SCALE ---N.T.S.</small>	<small>SH: 3 OF 5</small>

Figure 3-2. Elevation "F" and "G" of the Proposed Work Limits (From General Contractor).



COURTYARD FACADE
N.T.S.

MICHAEL K. KOEHLER P.C.		
<small>617 55 STREET, BROOKLYN, N.Y. 11220</small>		
FACE BRICK AND SHELF ANGLE/WINDOW LINTEL REPLACEMENT/LINTEL RESTORATION		
<small>215 PARK AVENUE SOUTH</small>		
<small>DRAWN BY: S.A.</small>	<small>CHECKED BY: M.K.K.</small>	
<small>DATE: 06-01-01</small>	<small>SCALE: ----N.T.S.</small>	<small>SH: 2 OF 4</small>

Figure 3-3. Elevation "H" of the Proposed Work Limits (From General Contractor).

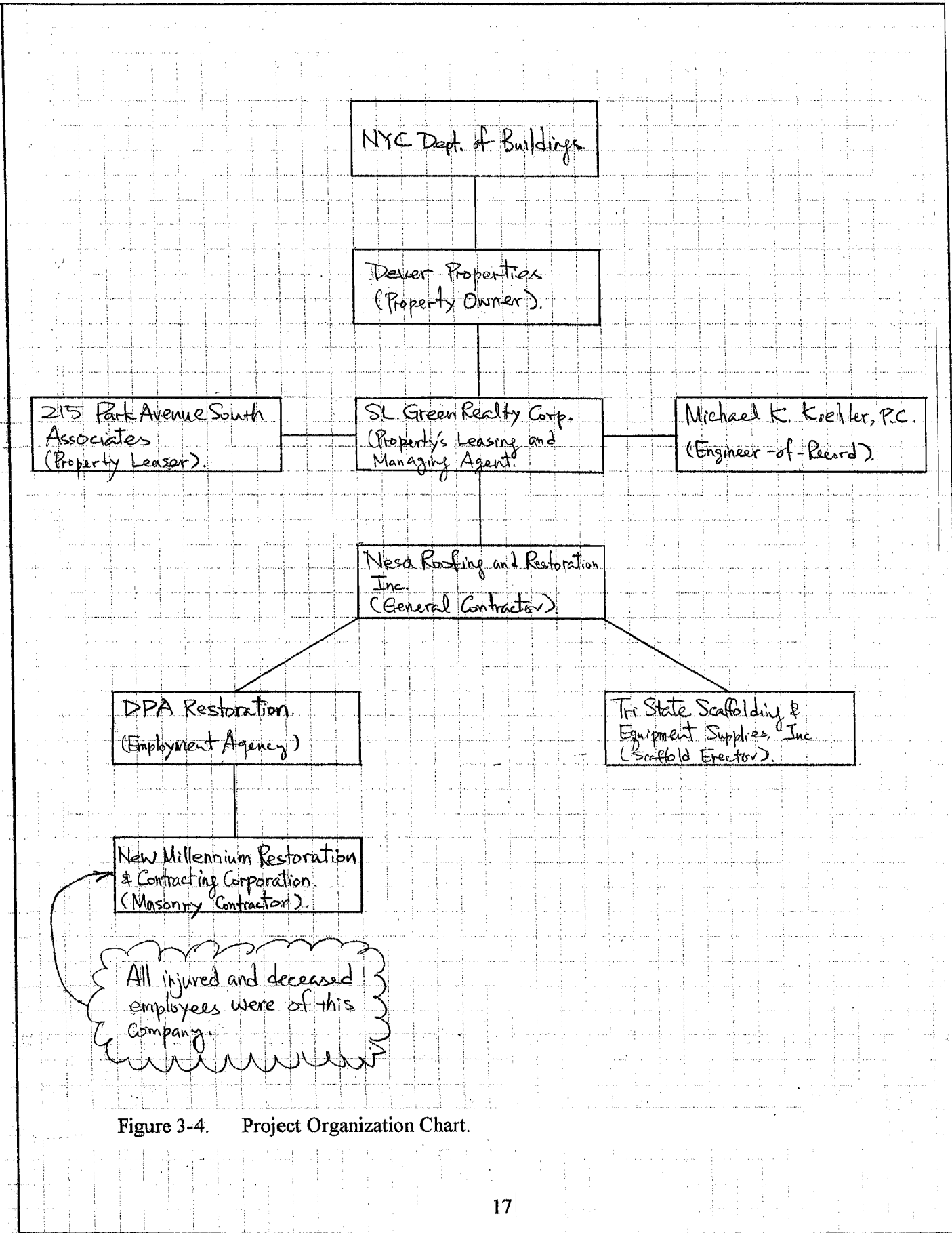
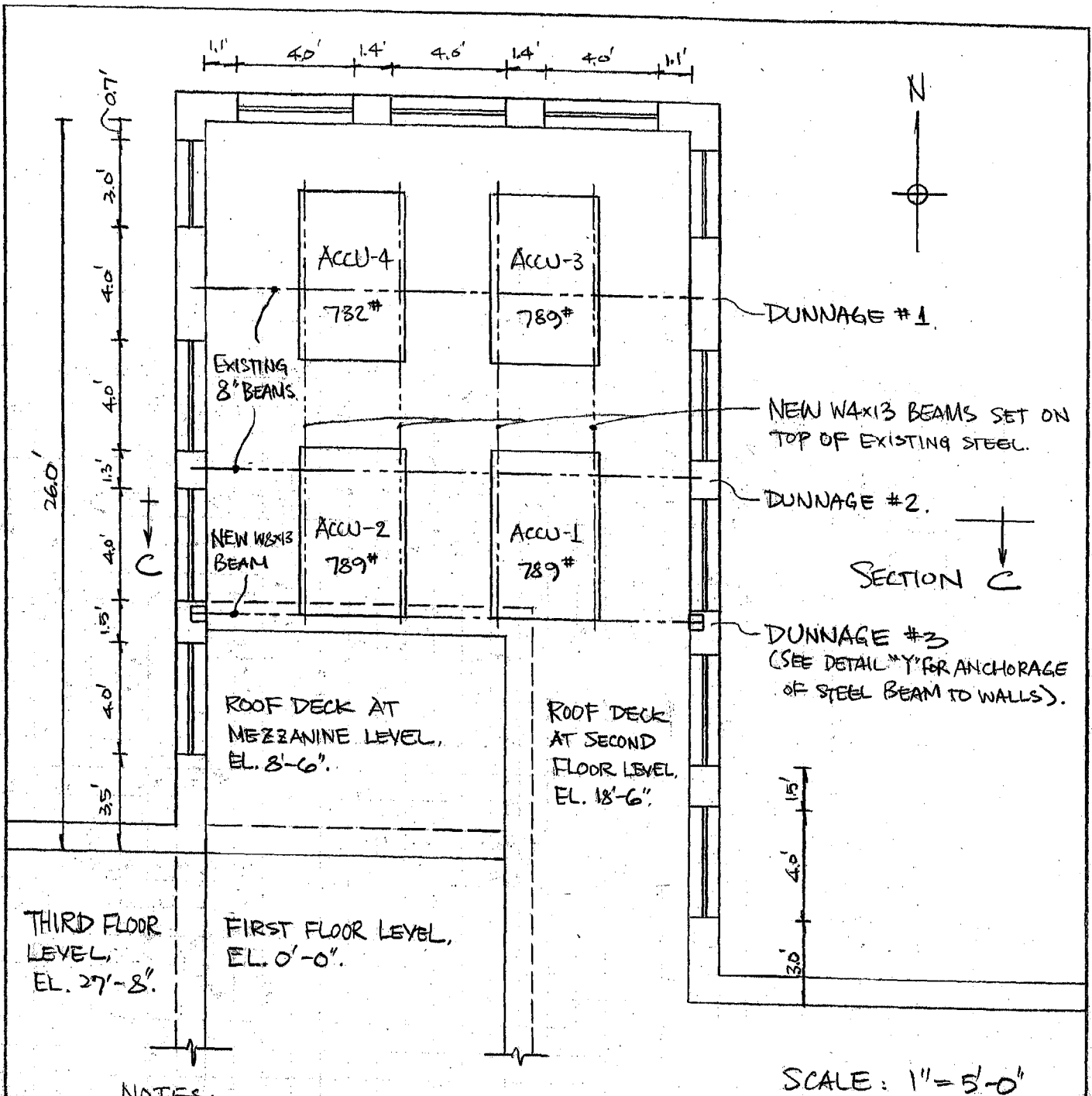


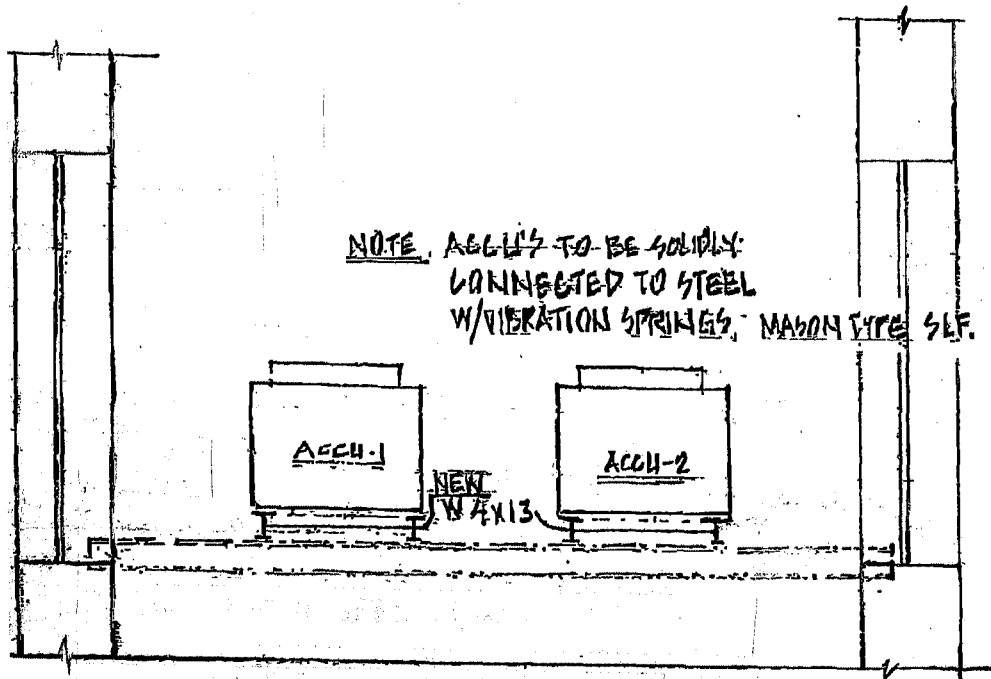
Figure 3-4. Project Organization Chart.



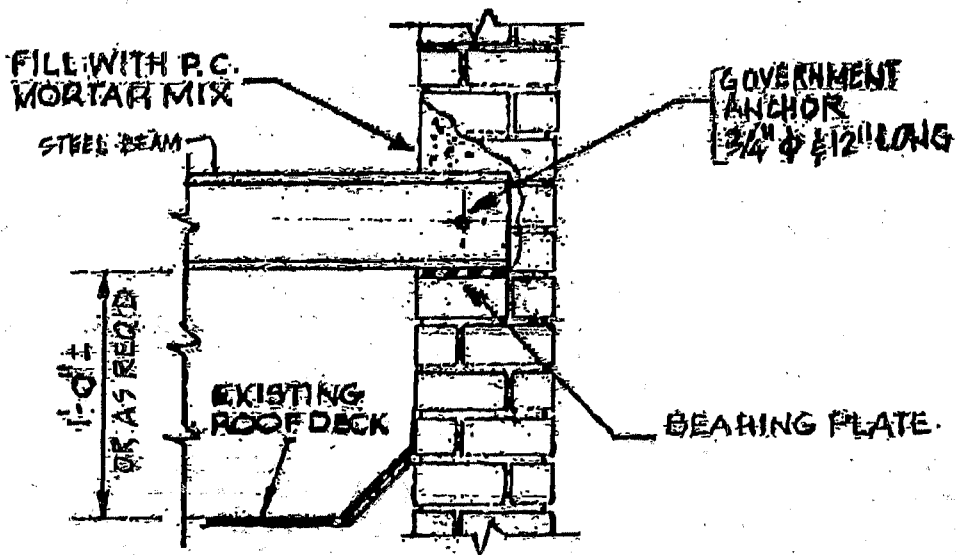
NOTES:

1. ALL NOTES AND CONFIGURATIONS ARE TRANSFERRED FROM ENGINEERING DRAWINGS PREPARED BY RICHARD HILL, P.C., DATED 12/18/91.

Figure 3-5. Plan View of the South Courtyard Prior to the Erection of the Scaffold Towers (Modified from Richard Hill, P.C., Drawing No. M-1, Dated 12/18/91).



SECTION C - C



DETAIL Y
NOT TO SCALE

Figure 3-6. Section C-C and Detail "Y" of Figure A-5 (Modified from Richard Hill, P.C., Drawing No. M-1, Dated 12/18/91).

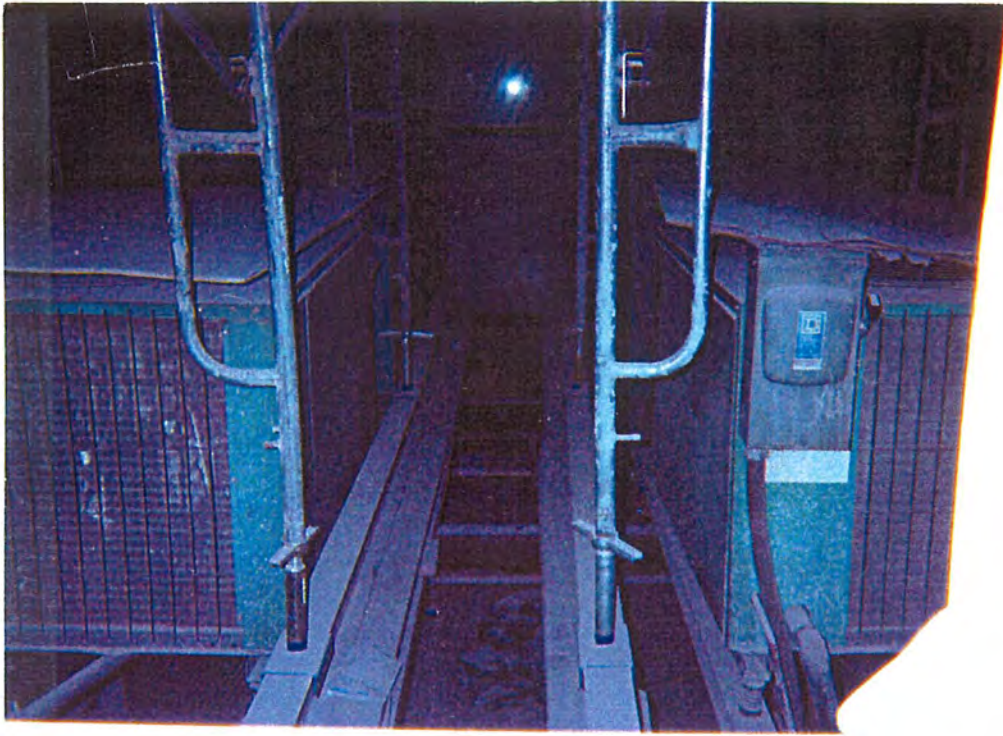


Figure 3-7. Support Conditions at the Base of the Northern Portion of the Scaffold Towers I (from Scaffold Erector, Looking Toward North).

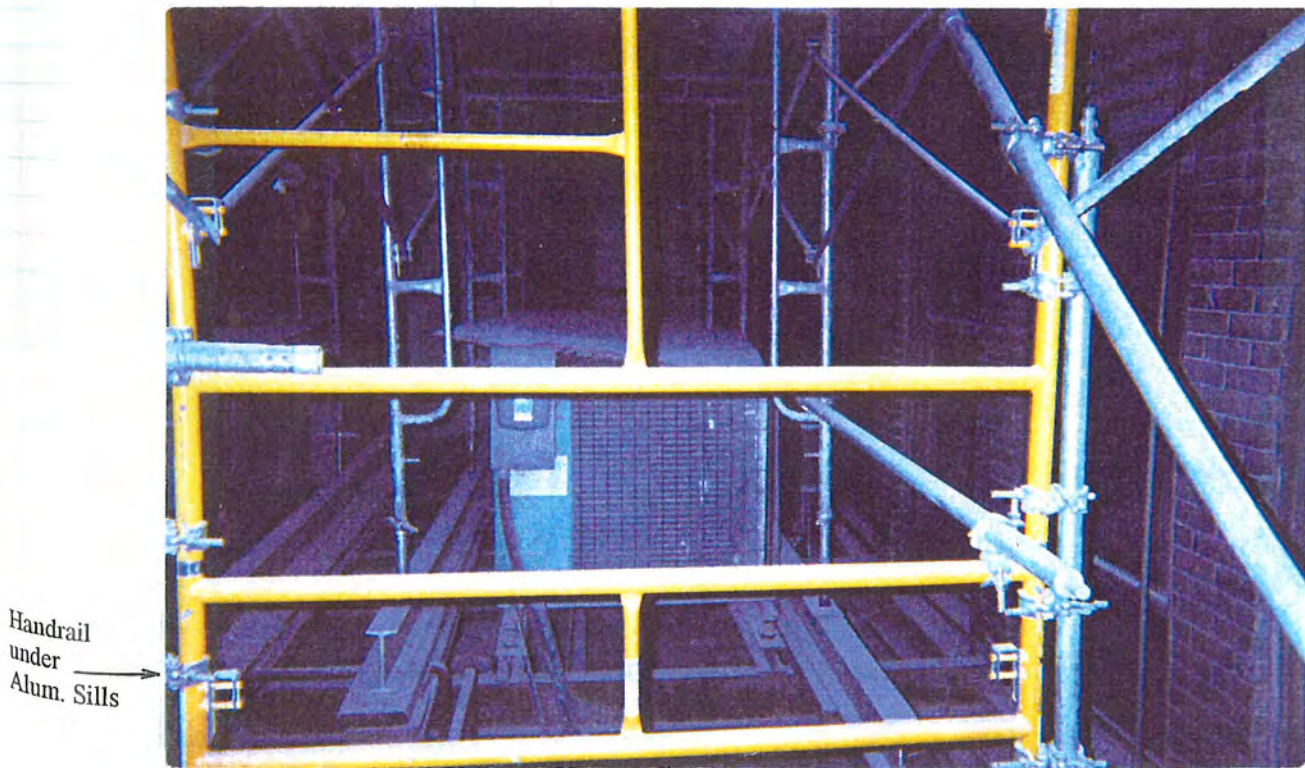


Figure 3-8. Support Conditions at the Base of the Northern Portion of the Scaffold Towers II (from Scaffold Erector, Looking Toward North).



Figure 3-9. Support Conditions at the Base of the Southern Portion of the Scaffold Towers I (from Scaffold Erector, Looking Toward Northwest).



Figure 3-10. Support Conditions at the Base of the Southern Portion of the Scaffold Towers II (from Scaffold Erector, Looking Toward North and Upward).

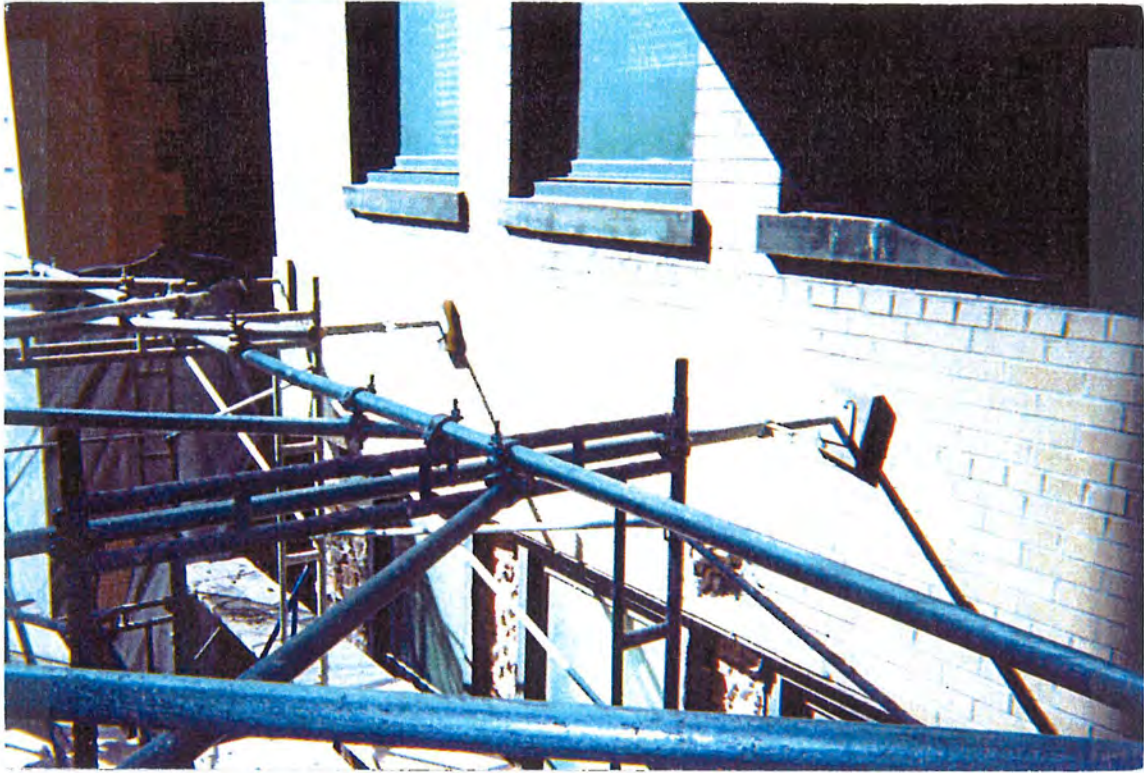


Figure 3-11. Lateral Bracing Conditions at the Top of the Scaffold Towers I (from Scaffold Erector, Looking Toward Southwest).



Figure 3-12. Lateral Bracing Conditions at the Top of the Scaffold Towers II (from Scaffold Erector, Looking Toward North). Note that the electrical hoist in the upper middle of this figure was mounted on top of the southmost bay of the east tower prior to the collapse.



Figure 3-13. Collapsed Condition of Dunnage No. 1 at West End.



Figure 3-14. Collapsed Condition of Dunnage No. 1 at East End.



Figure 3-15. Collapsed Condition of Dunnage No. 2 at West End.

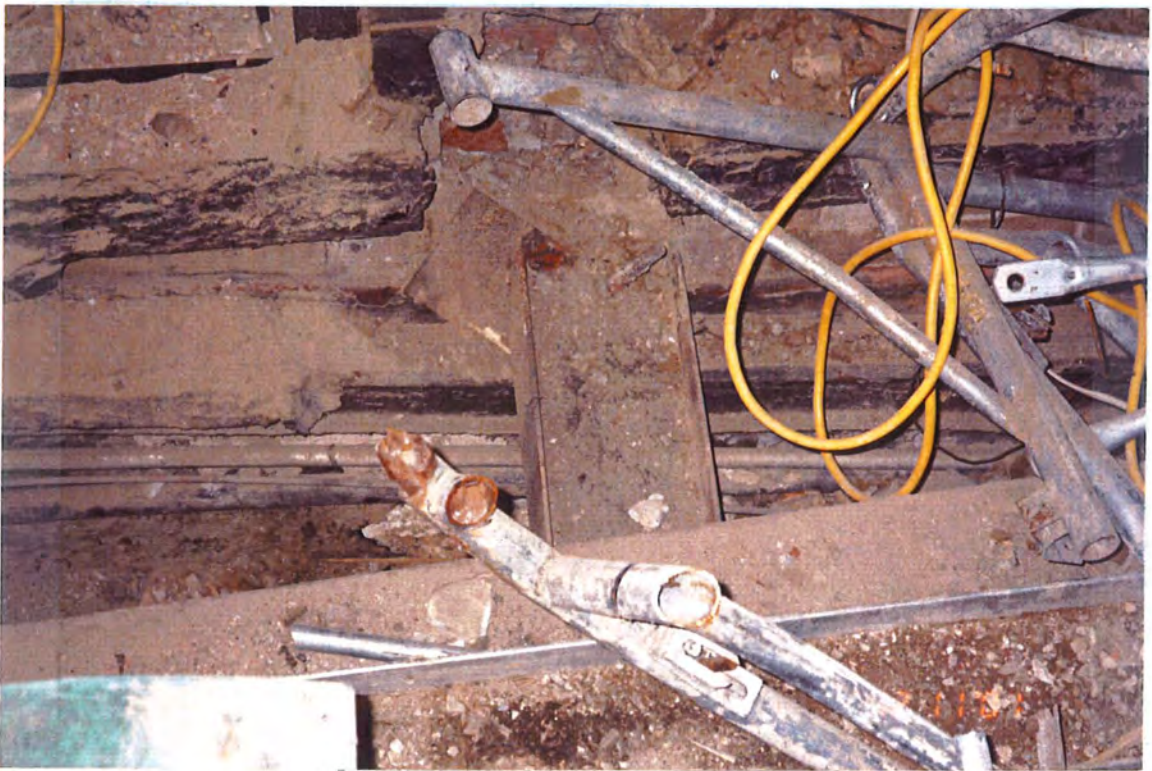


Figure 3-16. Collapsed Condition of Dunnage No. 2 at East End.



Figure 3-17. Collapsed Condition of Dunnage No. 3 at West End.

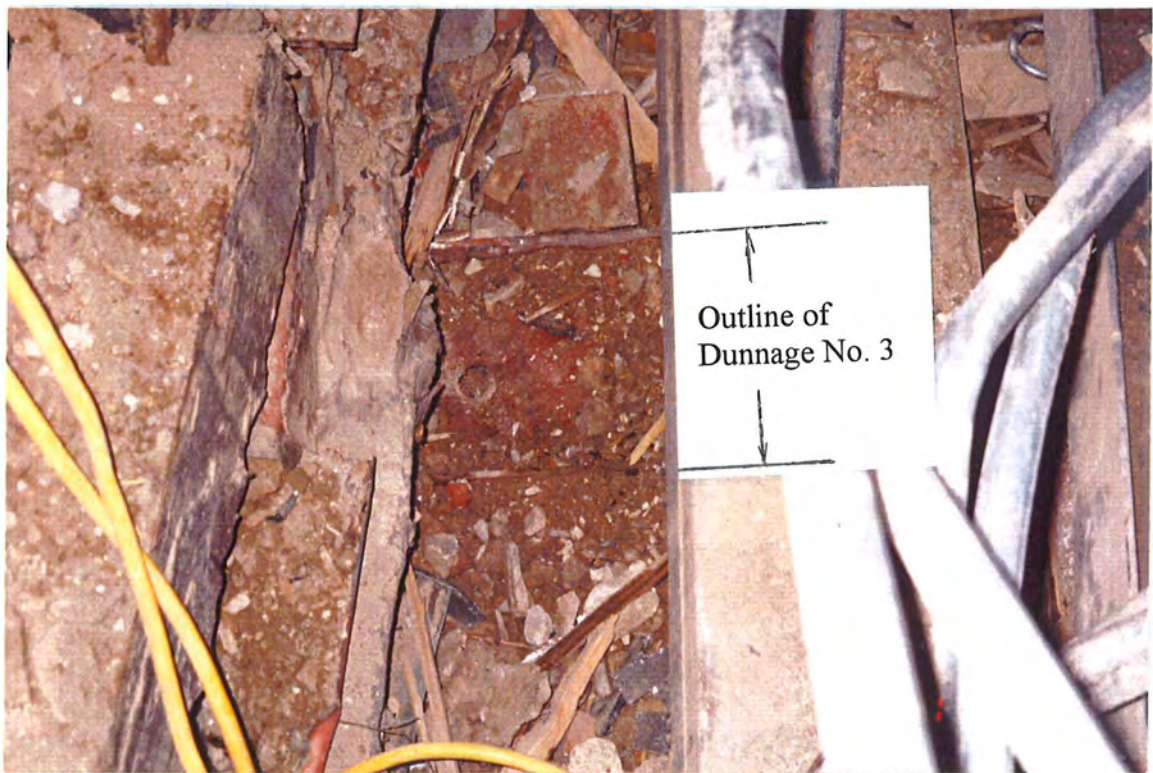


Figure 3-18. Collapsed Condition of Dunnage No. 3 at West End.

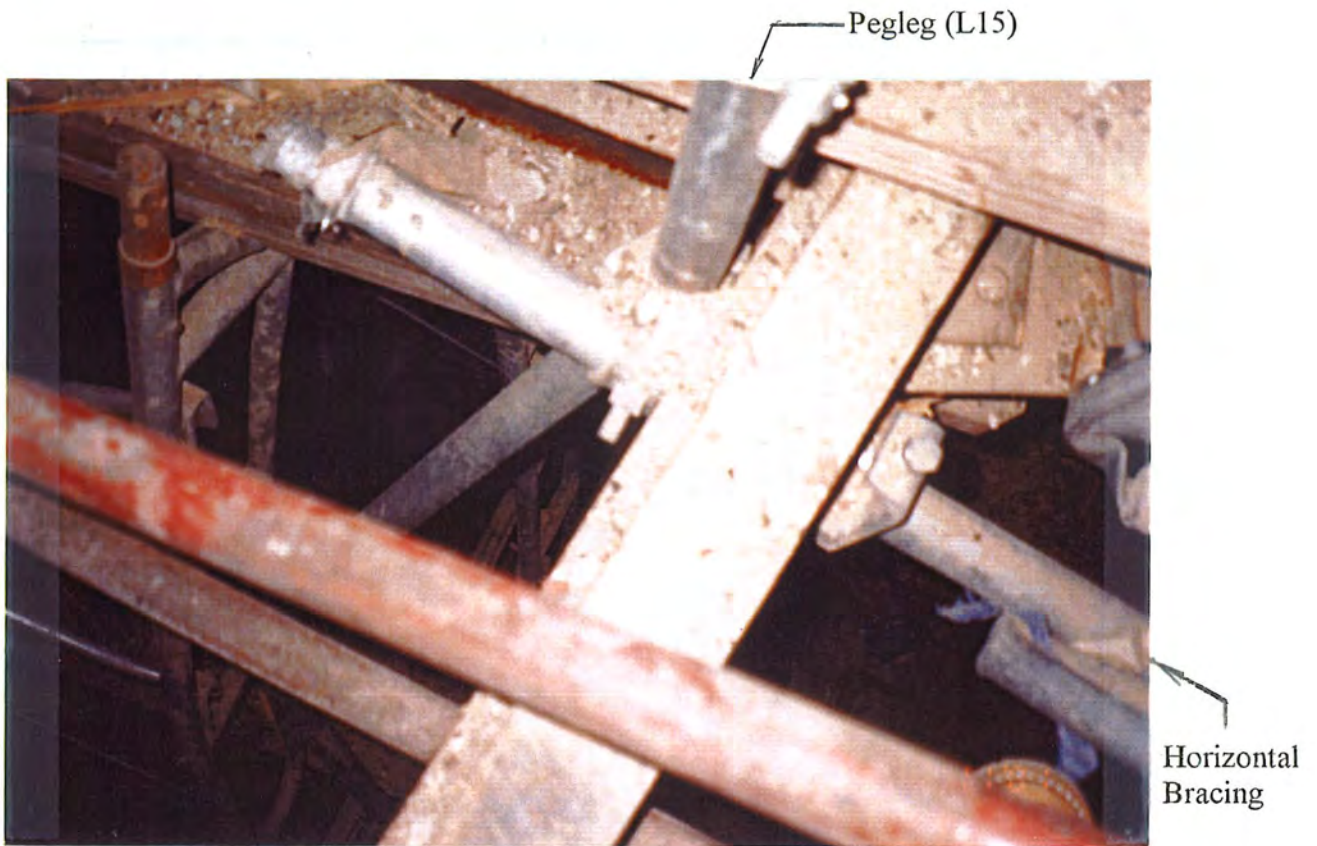


Figure 3-19. Failed Condition of the Pegleg with a Clear Bent at the Upper Bracing Connection (Looking Downward from Top).



Figure 3-20. Failed Condition of the Pegleg with no Bent at the Lower Connection to the Handrail (Looking Downward from Top).

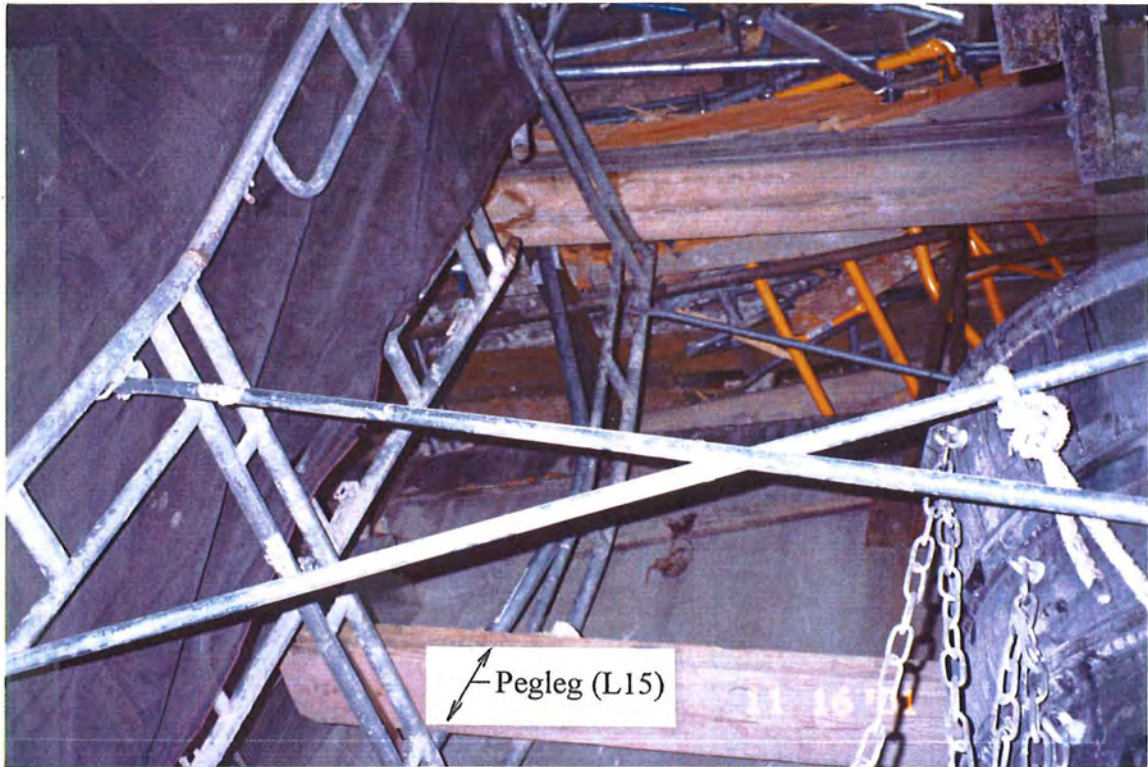


Figure 3-21. Failed Condition of the Pegleg (Looking Toward Northeast and Upward).

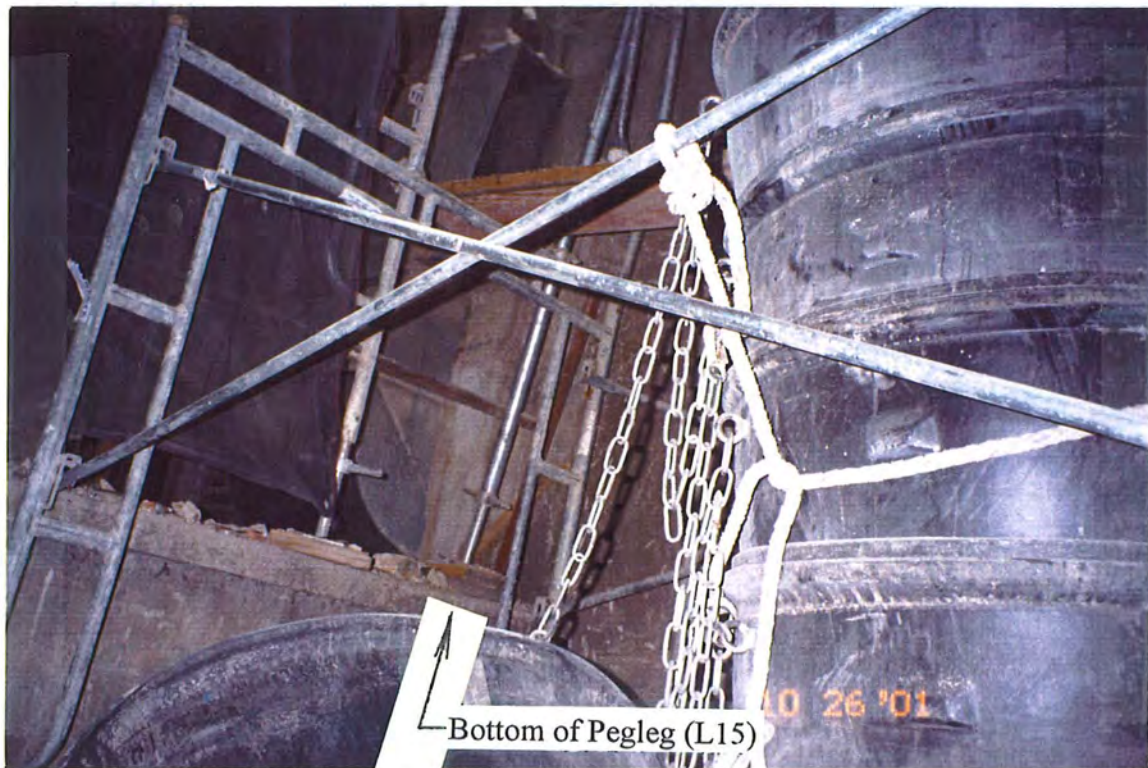


Figure 3-22. Failed Condition of the Pegleg with no Bent at the Lower Connection to the Handrail (Looking Toward Northeast and Upward).

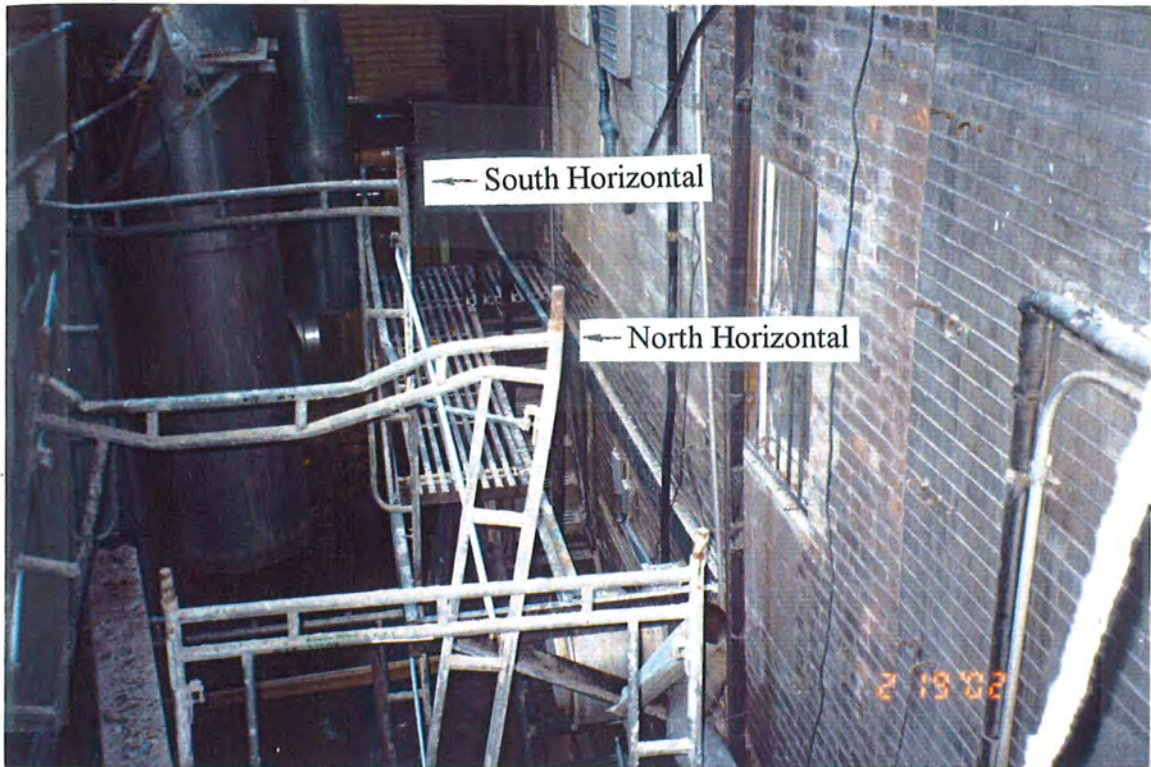


Figure 3-23. The Deflected Conditions of the Top Horizontal Members at the upper Frames of the Offset Scaffold from the First Floor Level (Looking Toward South).

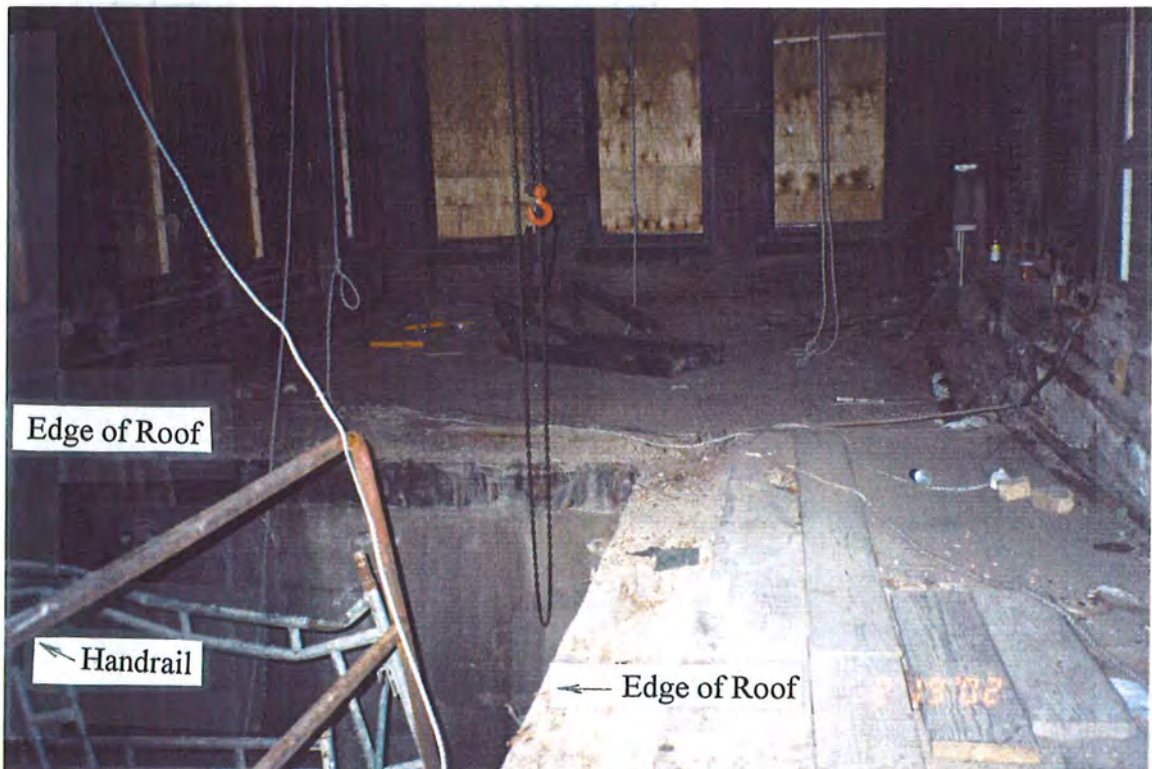


Figure 3-24. The Conditions of the Handrails along the Edges of the Roof Deck at the Second Floor Level after the Collapse (Looking Toward Northwest). Note that prior to the collapse the handrails were all around the edge of the roof and the pegleg was located near the intersection of two edges from the lower mezzanine level.



Figure 3-25. The Rotational Failure of Leg 17 above the South Top Horizontal Member of the Offset Scaffold (Looking Toward East).

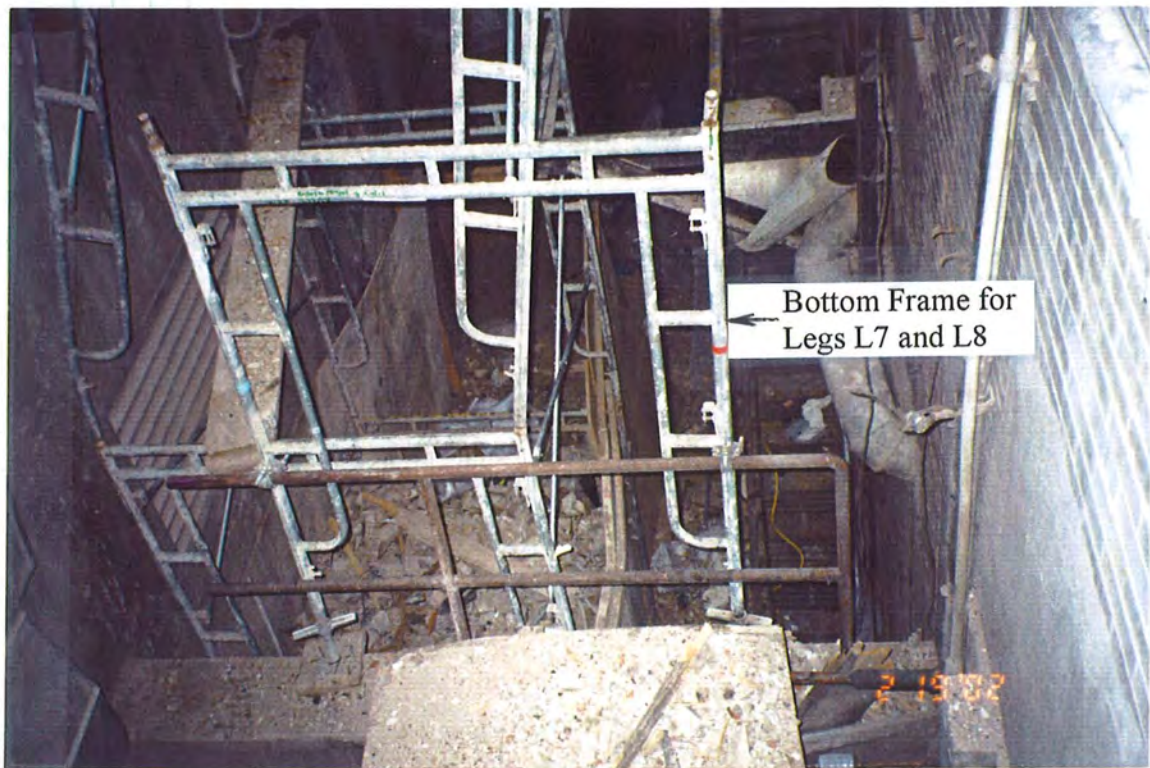


Figure 3-26. The improper Bracing Condition of the Bottom Frame from the Parapet Wall at the Mezzanine Level (Looking Toward South and Downward).



Figure 3-27. Condition of the Recovered Scaffold Frames, Lateral Bracings, and Bottom Support Members I.



Figure 3-28. Condition of the Recovered Scaffold Frames, Lateral Bracings, and Bottom Support Members II.



Figure 3-29. Condition of the Recovered Scaffold Frames, Lateral Bracings, and Bottom Support Members III.

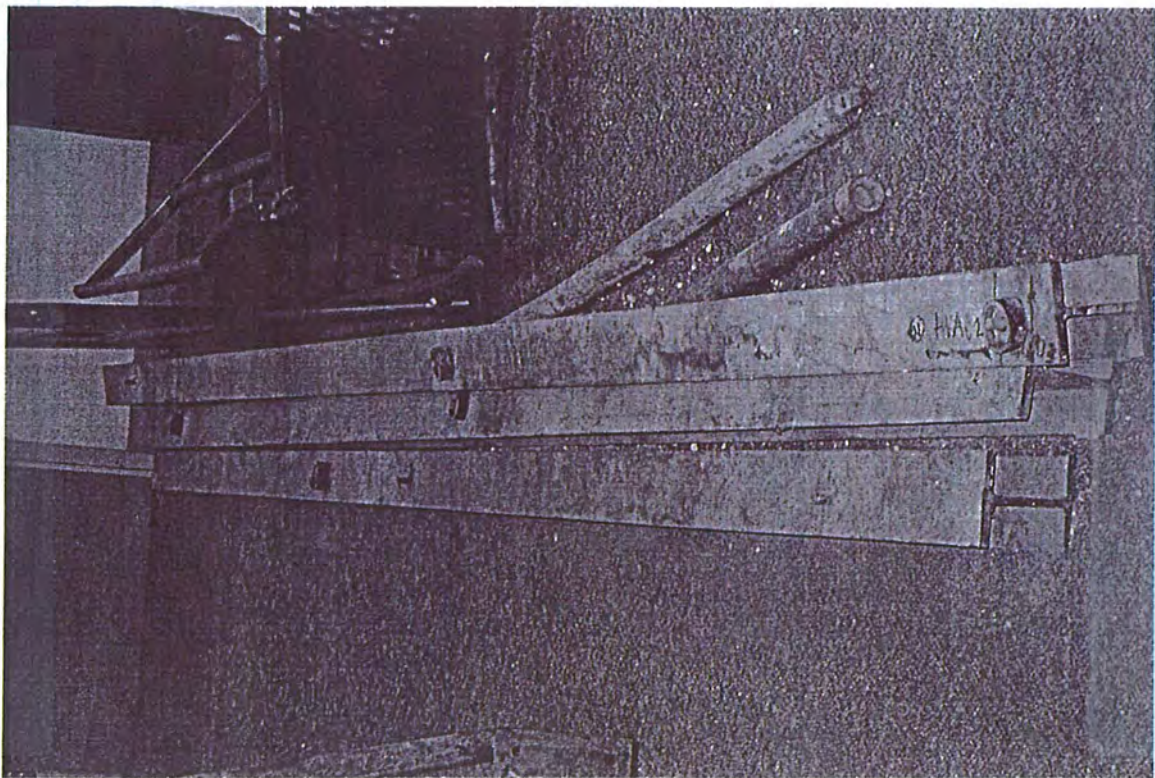


Figure 3-30. Intact Condition of the Other Three W4X13 Steel Beams.

APPENDIX A

ENGINEERING ANALYSIS

- A-1 Reconstruction of the Configuration of the Scaffold Towers
- A-2 Calculation of the Weight (Dead Load) of the Scaffold Towers
- A-3 Estimation of the Maximum Intended (Live) Load
- A-4 Estimation of the Actual Load on the Scaffold Towers at the Time of the Collapse
- A-5 Estimation of the Stress Level in the Main Support Members under the Scaffold Towers
 - A-5-1 Three 8-inch Steel Beams (Dunnage Nos. 1, 2 and 3) Supporting Legs 1 through 6 and 9 through 14
 - A-5-2 6-inch Steel Beam Supporting Leg 16
 - A-5-3 8-inch Aluminum Beam Supporting Legs 17 and 18
 - A-5-4 Lower 6-inch Steel Beam Supporting Both the Steel and the Aluminum Beams
 - A-5-5 4-inch Steel Beam (Dunnage No. 4) Supporting 8-inch Aluminum Beam
 - A-5-6 Offset Scaffold Supporting the Lower 6-inch Steel Beam
 - A-5-7 12-foot Long Pegleg Supporting Leg 15

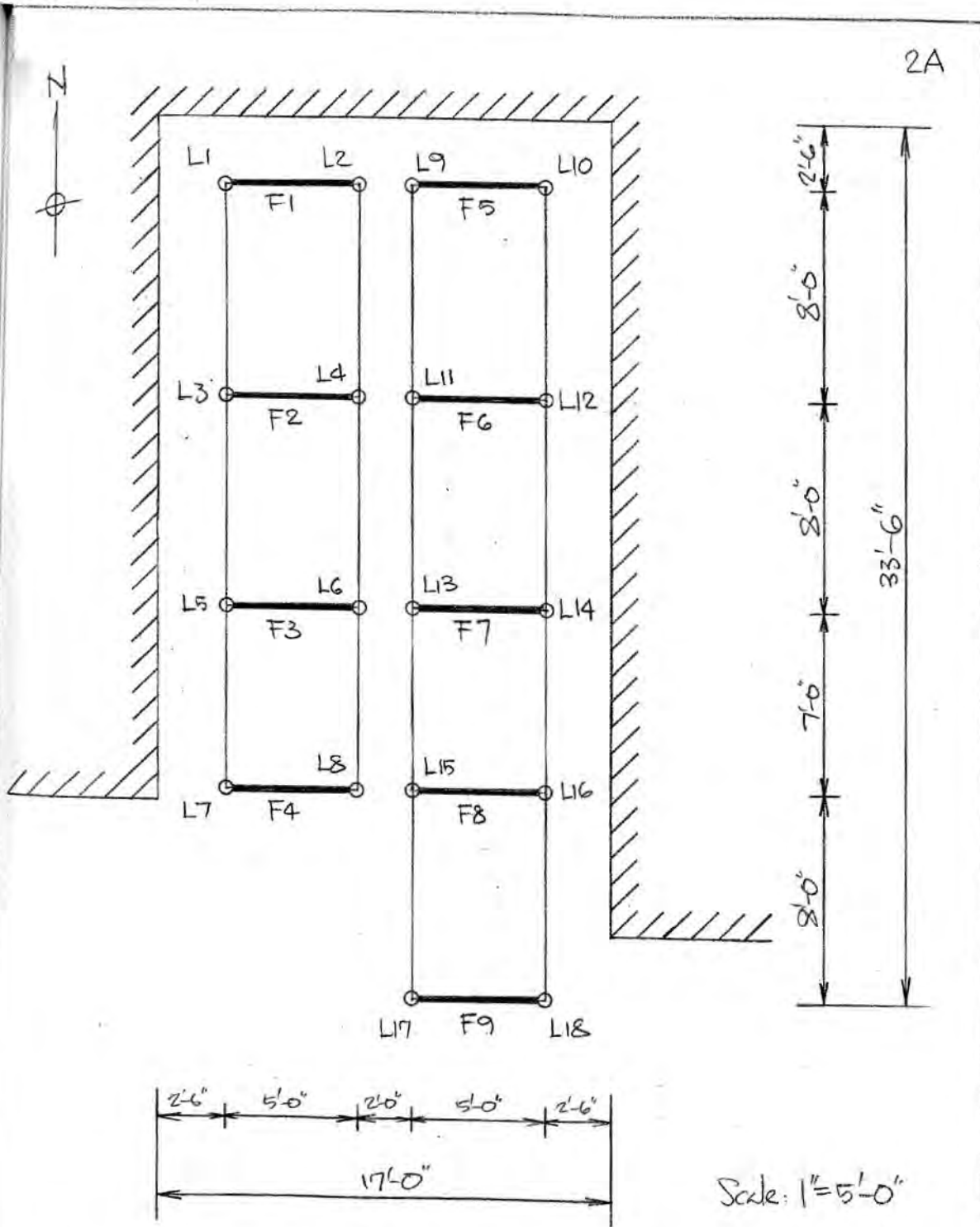


Figure A-1. Dimension and Identification of Main Scaffold Members in Plan.



Figure A-2. Two Scaffold Planks Stuck out on the South End of the East Tower at the 19th Level (from Scaffold Erector).



Figure A-3. Two More Levels, the 20th Level and the 21st Level of the East Tower Were above the Plank Stuck out Location (from Scaffold Erector).

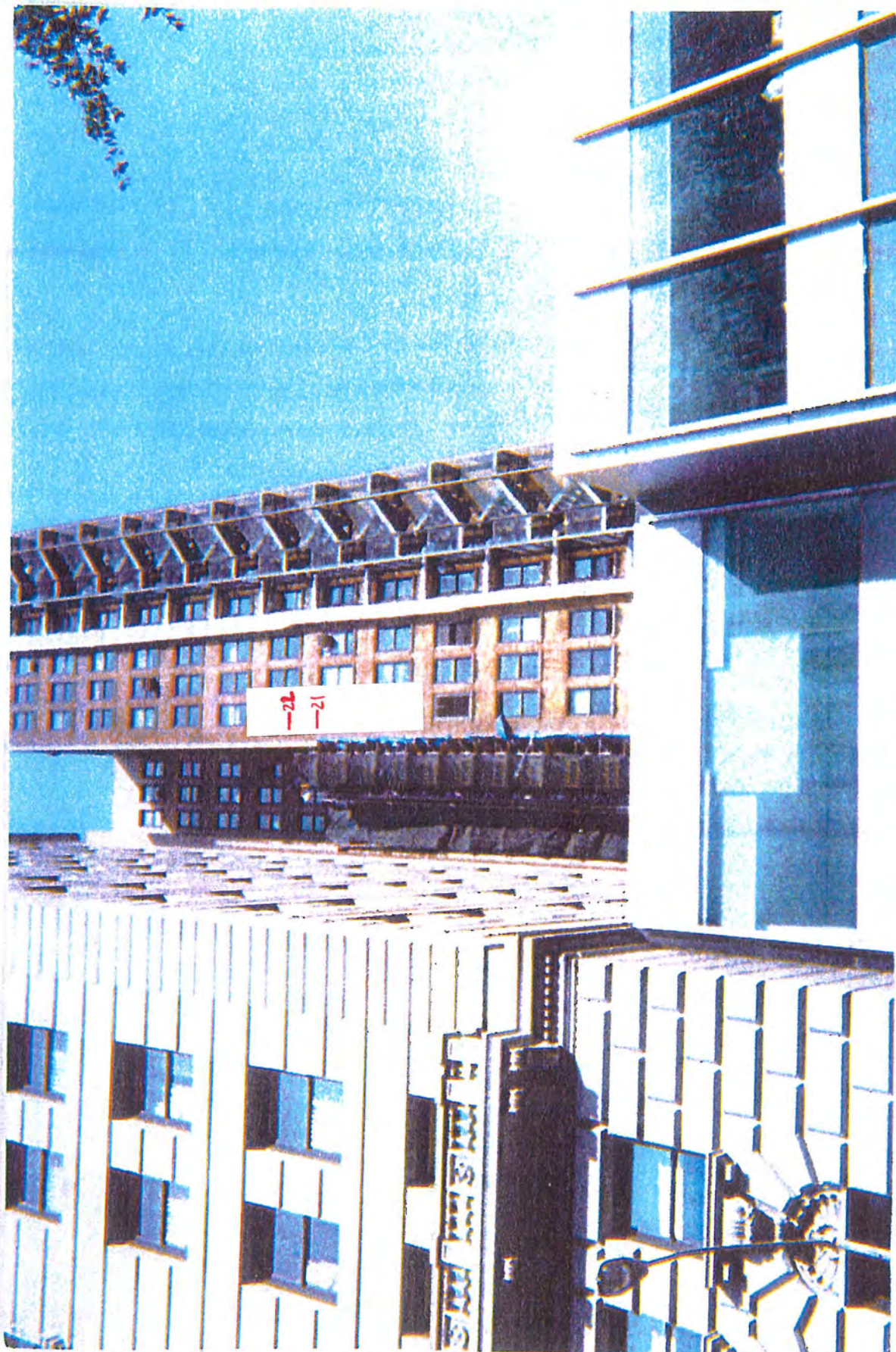


Figure A-4. One More Level, the 24th Level of the East Tower Was Added beyond Figure A-3 (from Scaffold Erector).