NBSIR 73-222

Investigation of the Skyline Plaza Collapse in Fairfax County, Virginia

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Center for Building Technology
Institute for Applied Technology
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U. S. DEPARTMENT OF COMMERCE, Frederick B. Dent, Secretary
NATIONAL BUREAU OF STANDARDS, Richard W. Roberts, Director

ABSTRACT

The collapse of the Skyline Plaza apartment building A-4 has been studied by using information contained in case records of the Occupational Safety and Health Administration (OSHA), U.S. Department of Labor and obtained from on-site inspections by investigators from the National Bureau of Standards.

Non-compliance with OSHA construction standards has been identified with regard to formwork, field-cured concrete specimens and crane installation. Specifically, the construction procedures did not comply with standards for the removal of supporting forms. It is concluded that premature removal of forms was a contributing factor to the collapse in building A-4.

An analysis of the 23rd-floor slab indicates that its most likely mode of failure was in shear around one or more columns in section 3 of the floor slab. The strength of the 23rd-floor slab on the day of collapse has been estimated to be at a level that removal of shoring could have produced shear failure in the slab.

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THE SKYLINE PLAZA COLLAPSE IN FAIRFAX COUNTY, VIRGINIA

by

Edgar V. Leyendecker and S. George Fattal*

1. INTRODUCTION

1.1 Background

The Skyline Center Complex located near Bailey's Crossroads, Fairfax County, Virginia is a development planned to contain eight apartment buildings, six office buildings, a hotel, and a shopping center [19].[1] Two apartment buildings which have been completed are shown in figure 1.1. A pair of apartment buildings (similar in appearance to those in figure 1.1) and an adjoining parking and lobby structure were under construction and included the structures which collapsed on March 2, 1973.

The apartment buildings under construction are shown in figure 1.2, an aerial photograph taken at about 11:00 a.m. on Friday, March 2, 1973, from an altitude of 5000 ft [18]. Several hours later, at about 2:30 p.m., a portion of the building shown in the top of figure 1.2 collapsed. The collapsed portion of the building was located approximately

* Dr. Edgar V. Leyendecker and Dr. S. George Fattal are structural research engineers with the Structures Section; Structures, Materials, and Life Safety Division; Center for Building Technology; Institute of Applied Technology; National Bureau of Standards.

[1] Numbers in brackets refer to references.

under the slab area being cast (the tall building in figure 1.2) and extended vertically for the full height of the building, 23 stories plus four basement stories. The collapse progressed horizontally from the tall building to include the entire parking garage area and stopped at the building partially shown at the bottom of figure 1.2. The full extent of the collapse is shown in figure 1.3. It has been reported by OSHA that fourteen construction workers were known to have been killed, four in the garage and ten in the tower, and another 34 injured in the incident.

Within a few hours of the incident an inspection team from the Occupational Safety and Health Administration (OSHA), Department of Labor, began arriving at the site to begin an investigation into the collapse.

1.2 Objective and Scope

On Monday, March 5, OSHA requested the technical assistance of the Center for Building Technology of the National Bureau of Standards (NBS) with respect to the collapse. The National Bureau of Standards was requested to ascertain, if possible, the cause of the incident, to assist OSHA in determining whether there had been non-compliance with OSHA standards ("Safety and Health Regulations for Construction" [10]) and whether such non-compliance contributed to the collapse.

OSHA compliance officers were on the site from March 2 through March 16 collecting material for their case records

[18]. During this time, personnel from the NBS made numerous site inspections.

The NBS investigators used data gathered during on-site inspections, OSHA case records, structural and architectural drawings, shop drawings, and structural computations in preparing this report. Where reference is made in this report to employee statements, such statements are part of OSHA case records examined by NBS.

2. DESCRIPTION OF THE STRUCTURES AND THEIR COLLAPSE

2.1 Introduction

Three structures may be identified in figure 1.2; building A-4 at the top, building A-5 partially shown at the bottom, and a parking garage in between the two buildings. The collapse started in building A-4 and progressed vertically to the ground and horizontally to include the entire parking garage. The collapse stopped at building A-5 which was structurally isolated from the parking garage. The two affected structures are shown in a plan view in figure 2.1 and are discussed in subsequent sections.

The three structures under construction were designed under the Fairfax County Building Code Ordinance [5] which incorporates by reference the provisions of the American Concrete Institute's Building Code Requirements for Reinforced Concrete (ACI 318-63) [6]. The building design is discussed further in section 4, Structural Investigation of Failure Conditions in Building A-4. Applicable Federal safety regulations are described in the Safety and Health Regulations for Construction [10] which incorporates by reference the American National Standards Institute's Safety Requirements for Concrete Construction and Masonry Work, ANSI A10.9-1970 [2].

2.2 Building A-4

2.2.1 General

Building A-4 was of reinforced concrete flat plate construction supported on a 4-ft thick foundation mat. The completed structure was to have 26 stories of apartments, plus a penthouse and a four-story basement (designated B-1, B-2, B-3, and B-4). The typical story height from the first story up was 9 ft 0 in from top of slab to top of slab. Floor slabs were 8 inches thick.

The basement story heights varied in order to suit mechanical equipment layout with a total basement height of about 40 ft as measured from the top of the foundation mat (level B-4) to the top of the first floor slab. The basement floors were about the same elevation as the corresponding floors in the garage, although there was no actual floor B-3 in building A-4. That is, the lowest "story" in the basement consisted of stories B-4 and B-3. The first floor slab was at the same elevation as the roof of the garage which was to become a landscaped area.

The plan view shown in figure 2.1 is that of the 22nd story and is typical for the 1st through 26th stories in the column layout. The column layout remained essentially the same through the height of the building (1st story and up) varying only in size, reinforcing steel, and concrete strength. There are eight shear walls in the structure. These are designated as A through H in the drawing. The configuration of the shear walls above the 20th story is shown in figure 2.1. For the 20th story and below, the portion shown dashed became part of the shear wall. The floor slab thickness of 8 in was constant through the height of the building. A 1/2-in expansion joint separated the building into two parts at grid line H.

Normal weight aggregate concrete was used in the columns. The specified column concrete strength varied; it was 5000 psi from the foundation mat to the 7th floor, 4000 psi from the 7th to the 17th floor, and 3000 psi above the 17th floor. The slabs used lightweight aggregate (coarse aggregate only) concrete with a specified strength of 3000 psi. Inspection of two floors, the 24th and 10th, indicated that the lightweight aggregate concrete floor slab passed through the columns at these floors.

A typical floor of the building was poured in four sections; the progress of construction at the time of the collapse is shown in figure 2.2 [18]. The actual sizes of the pour sections are shown in figure 2.1. Note that the sections are not equal in size and configuration. According to statements provided to OSHA, a normal construction rate was one section a day. This rate of construction would permit the completion of one floor per week and allow an extra day for weather variation. In actual practice, this rate was not always maintained as indicated in figure 2.3.

Figure 2.3 is a plot of daily temperature versus calendar date for January 28 through March 2, the day of the collapse. The maximum, minimum, and average temperatures are those recorded at Washington National Airport, the official weather bureau recording station nearest the construction site. Where available, the daily temperature range recorded at the construction site (from job records) is also plotted. The dates for pouring the various sections of the 21st through 24th floors are indicated on the drawing. Note that the casting dates shown tended to fall on the days with the higher temperatures.

The general appearance of the building A-4 as viewed from the southeast after the collapse is shown in figure 2.4. A number of the floors and one of the columns is identified for later reference. Note the absence of a floor slab at level B-3 (in accordance with the plans). Note also the column corbels at floor levels B-3, B-2, B-1, and 1. These corbels supported one edge of the parking garage slabs. Level B-4 of the garage was on grade.

A partial view of the north face of building A-4 is shown in figure 2.5. A number of columns and floors are identified. The collapse extended between shear wall H and column 33 on the south face, a distance of about 65 ft (refer to figure 2.1). On the north face the collapse extended between columns 12 and 17, a distance of about 104 ft. Note that for most floors the slab between columns 16 and 17 did not sever from the main building but is sagging for a one-story height. Below the 20th floor, the horizontal extent of the collapse is as discussed above. Above the 20th floor, the failure zone extended slightly to the west and a greater amount to the east.

Figures 2.6 through 2.8 are closeup views of the east end of the failure zone. These figures require little comment at this time; however, it should be noted that the formwork is clearly visible in figure 2.6.

A general view of the west end of the failure zone is shown in figure 2.9 along with closeup views in figures 2.10 through 2.12. These figures also require little comment at this time; however, it should be noted that the formwork is clearly visible in figure 2.10.

On the night of March 4 the remaining portion of the building to the east of the failure zone was completely demolished, with the result shown in figures 2.13 and 2.14.

2.2.2 Formwork and Interior Views

Access was not gained to building A-4 by OSHA personnel until March 5 when the building appeared as in figure 2.14. Most of the photographs used in this discussion of the building interior were taken on or after that date. Between March 2 and March 5, additional shoring was placed in some parts of the building.

Figure 2.15 shows the locations of the formwork on March 2, shortly after the collapse. Full formwork was in place on the 24th and 23rd stories in sections 1 and 2 (refer to figure 2.1 for location). Some formwork may be seen on the 22nd story in sections 2 and 3. In section 2 this is primarily the area around the material elevator shown toward the western end of the south elevation in figure 2.15. Investigation by OSHA personnel on March 5 indicates that the only formwork not stripped on the 22nd story in section 2 was around that elevator. Employee statements indicate that this area was not stripped in order to prevent lumber from falling on bricklayers working below. Formwork may also be seen in section 3 of the 22nd story in figure 2.15. However, closer examination of the 22nd story in figure 2.10 indicates that formwork was stripped in the center portion of the 22nd story.

Full formwork was also in place under section 4 on the 22nd and 23rd stories. No reshoring can be seen under section 4 below the 22nd floor in figure 2.15 or in figures 2.4, 2.5, and 2.6. Some employee statements indicate, however, that reshoring was present below pour 4 on the 21st story. Other statements contradict this.

There is conflict also among employee statements as to the formwork which was in place in section 3. Employees interviewed by OSHA agreed that the 23rd-story forms supporting the recently cast 24th floor were in place. However, employee statements indicate that the 22nd-story forms were: (1) entirely removed, (2) partially removed, or (3) not removed. The same contradictions were indicated for reshoring on the 21st story. The angle of the aerial photograph in figure 1.1 prevents examination of conditions of formwork in section 3. The location of formwork will be discussed further after a description of the interior of the building.

The formwork on the 24th story is shown in figure 2.16. Several columns have been identified for reference (see figure 2.1 for location). These slab forms had not been completely erected. A schematic of this formwork is shown in figure 2.17 as derived from OSHA case records [18].

The formwork in figure 2.17 is typical for the floor slabs with design noted by the concrete contractor as being based on the publication, Formwork for Concrete, SP-4 [11]. Formwork sheets submitted to Fairfax County [18] contained no mention of lateral bracing for the form system. However, a limited amount of bracing may be seen in figure 2.16 and some subsequent figures. OSHA regulations (ANSI-A10.9, Sections 6.3.2 and 8.1.5) [2][2] require a lateral bracing system capable of resisting a lateral force of at least 2 percent of the dead load of the slab.

As may be seen in figure 2.16, the stringers were placed east-west and the joists were placed north-south. Employee statements indicate that stringers and joists were usually 16 ft in length although some were as short as 6 to 8 ft.

The erection procedure is described by reference to figure 2.16. Stringers were erected with a shore under each end (shores 1 and 5). These shores were attached to the stringers by a metal or wooden plate. Shores 2, 3, and 4 were then inserted in basket-like sockets which were fastened to the stringers. Employee statements indicate that diagonal bracing was installed in the north-south direction on every 4th shore at approximately 16-ft intervals (also see figure 2.20). The joists were placed over the stringers at about 16-in intervals (figure 2.17 indicates the spacing could be 20 in). Joists were toenailed to the stringers at about 8-ft intervals. Plywood

[2] For relationship between OSHA regulations and ANSI-A10.9 see section 2.1.

sheathing was then placed on the joists with the long dimension parallel to the stringers.

Forms were removed by knocking out shores 1, 2, 4, and 5 in figure 2.16. Shore no. 3 would then be knocked down which would allow the stringer to fall. The joists and plywood would then be pulled down.

Usually, the center portion of the slab would be stripped first. Approximately 10 stringers would be pulled down before reshoring began. The area stripped prior to reshoring was approximately 20 ft wide by 35 ft long.

During the NBS site inspection, a considerable portion of the lumber used for the remaining forms in the 24th story was found to be in poor condition or out of plumb. For instance, the top photograph in figure 2.18 shows a stringer with a battered end directly over the shore. The second photograph illustrates a shore which is out of plumb and has a vertical crack near the top. OSHA regulations (ANSI-A10.9, sections 8.1.24 and 8.1.25) [2] require correction of both of these conditions prior to placing concrete (which was not yet placed on these forms).

The portion of section 3 of the 24th-story slab which remained standing is shown in figure 2.19. This portion of the slab was several hours old at the time of the collapse. Apparently, the only activity on the slab at the time of collapse was work being done by concrete finishers. One concrete finisher working near the stair well (around columns 65 and 66) indicated that a large deflection was seen in the middle of section 3. The statements of numerous employees agree on this point. No statements provided an exact location of the sag. However, the middle of section 3 is about midway between columns 67 and 85 (refer to figure 2.1 for location).

The formwork on the 23rd story is shown in figure 2.20. These forms supported the 24th floor which had been cast on February 28 (section 1) and March 1 (section 2). Note the number of shores out of plumb and the location of lateral bracing. The method of attaching the brace to the stringer may be seen in figure 2.20. The method of attaching the brace to the floor is shown in figure 2.21. A portion of a nominal 3 x 4 was nailed to a piece of plywood which was in turn nailed to the floor. A plywood plate was occasionally used to nail the brace to the nominal 3 x 4. Note the absence of such a plate in figure 2.21. Occasionally, the brace was installed with the plate, in other instances, there was no positive attachment of the brace to the floor. In one case, the plywood stop was not nailed to the floor, providing no lateral resistance.

As in the case of the 24th story, the 23rd story lumber which was apparently damaged was reused. Examples of battered shores are shown in figure 2.22. Note the large reduction in bearing area. As in the case of the 24th story, OSHA regulations (ANSI-A10.9, sections 8.1.24 and 8.1.25) [2] required correction of these conditions prior to placing concrete.

Reshoring in section 2 of the 22nd story as of March 6 is shown in figure 2.23. OSHA investigators have indicated that the reshoring on March 5 was not as extensive as shown in figure 2.23.

Apparently on March 5 the reshores were principally in place on the balconies, as they were on March 3, the day of the collapse.

Careful examination of figure 2.23 reveals that the center ceiling area of section 3 (area beyond column 59) was bare, indicating that some stripping had occurred in section 3 (see also figure 2.10). Although there is conflict in their statements, a number of workmen that escaped the building by way of the stairs in section 4 (see figure 2.1) have indicated that the 22nd story was either partially or entirely stripped in section 3. Notes on the engineer's structural drawings[3] call for two full stories of shoring and one story of reshoring under a slab being cast.

A view of a portion of section 2 of the 21st story is shown in figure 2.24. Column 59 is at the expansion joint (Grid H in figure 2.1). Note the cables on the floor which were installed after the collapse to anchor to section 2 the portion of section 3 which was still standing. No reshoring was present on this floor in sections 1 or 2.

A number of workmen have stated that at least some reshoring was present in sections 3 and 4 of the 21st story. The statement of one employee indicates that he was working in section 3 installing reshores. At about 2:00 p.m. he could hear the stripping crew working on the next floor up (22nd floor). He could hear the noise caused by the falling lumber. At about this same time, the workman's reshores began falling so

[3] Skyline Center Structural Drawings, Weihe, Black and Jeffries, Architects, dated March 16, 1972, with revisions through October 2, 1972 - Drawing S5.

that just before the accident there were no reshores in section 3 (this point is discussed later).

As shown in figure 2.25, the 20th floor was completely bare of reshoring. This was reportedly the case for sections 1 through 4. Note the presence of shear walls A and B.. These walls were not present (the design did not require them) above the 20th story. Note also the erection crane which is barely visible to the left of the designation for shear wall A. A view of the failure section at the end of the 20th floor is shown in figure 2.26.

It has been indicated in this report that there is conflict in the various employees' statements as to what formwork was in place at the time of the incident. However, it should be noted that conditions changed throughout the day as construction proceeded; that is, what may have existed at one time may not have existed a short while later. Nevertheless, it is necessary to determine the location of forms and reshores at the time of the incident.

The estimated location of forms and reshores is given in figure 2.27 and discussed below. Physical evidence examined by OSHA and NBS investigators as well as employees' statements indicate that all formwork was in place for all poured sections on the 23rd and 24th stories. Based on employees' statements and careful observation of photographs taken on the day of the collapse and over the following days, it is concluded that all formwork had been removed from sections 1 and 2 of the 22nd story on the day of collapse. The one exception is in the limited area near the materials elevator on the south face of the building. It appears that very little reshoring had been done. Examination of photographs indicates that formwork was in place in section 4 of the 22nd story. Section 3 is discussed later. On the 21st story there was no reshoring in sections 1 and 2. One statement indicates that the reshoring had been removed from section 2 on the day of the accident. Although some statements indicate the presence of reshores in section 4 of the 21st floor, this is not supported by photographic evidence available (figures 2.4, 2.5, 2.6, and 2.15). The photographs at the angle at which they were taken would not show reshoring, if any, located at some distance away from the edge of the slab. It is considered possible that there was some reshoring present although probably a small amount. Section 3 of the 21st story is discussed below.

One workman indicated that, at the time of the incident, he was placing reshores in section 3 of the 21st story and that some reshores were present when he started work. Prior to the incident all of the reshores fell out (except those in the balcony areas). This is consistent with what could occur if the forms had been removed in the story above. For example, consider figure 2.28 (a) which shows a frame with forms in the 3rd and 4th stories and reshores in the 2nd story. The fifth floor has been freshly poured and carries none of its own weight which must be distributed to the 4th, 3rd, and 2nd floors. The exact distribution of loads depends on a number of factors, one of which is the construction history.

If the forms are removed in the 3rd story as in figure 2.28 (b), then the 4th floor carries all of its own weight in addition to that of the fifth floor. The 2nd and 3rd floors are now relieved of the previous load from the 4th and 5th floors. The 3rd floor deflection will decrease due to this reduction of load. A possible smaller contribution to the decrease in the 3rd floor deflection is due to the downward deflection of the 4th floor under the added load causing the third story column to bend outward and the 3rd floor to bend upward as schematically illustrated in figure 2.28 (c). The net result of the removal of forms causing an upward deflection of the 3rd floor could quite conceivably cause the 2nd story reshores to fall out with the final result shown in figure 2.28 (d).

Based on the above simplified analogy plus previously mentioned photographs (figures 2.10, 2.15, 2.23) it is concluded that form stripping was in progress in the 3rd section of the 22nd story. The photographic evidence indicates that at least the central portion of the section was being stripped heading toward section 4.

2.2.3 Erection Cranes

Two climbing cranes were used in the construction of building A-4. Crane no. 1 was located in section 2 and crane no. 2 was located in section 4. The terminology used in the discussion of the cranes is shown in figure 2.29 [22].

Both cranes were initially used in a free-standing position mounted to the foundation mat. Later, as construction proceeded, the cranes were supported by floors within the building. Except for certain installation requirements, crane dimensions, and capacity, the two cranes were quite similar. Descriptive data are contained in figures 2.30 [21] and 2.31 [22] for the two cranes.

The standard climbing crane consists of the tower base section (which contains the climbing machinery, no. 2 in figure 2.29), four standard tower sections (no. 3 in figure 2.29), and a tower slipring section (no. 4 in figure 2.29). The four standard tower sections are equal in length and may be interchanged. The ring gear assembly (no. 5 in figure 2.29) which forms the connection between the stationary tower and the rotating portion of the crane fits on top of the slip ring section.

A schematic of the climbing sequence is shown in figure 2.32 [22]. In picture 1 the crane is installed in its initial position on the foundation mat. A lower support, the climbing frame, is installed a maximum of 36 ft from the mat. The climbing ladder is suspended from the climbing frame (the ladder is about 38 ft long). An upper support, consisting of four corner clamps, is attached a specified distance above the lower support. This required distance was specified as 21'-4" for crane no. 2 (figure 2.31) and 18'-4" for crane no. 1 (figure 2.30). In the actual use, a two-story distance of 18'-0" was used for crane no. 1 and apparently for crane no. 2. In picture 2, the tower climbs up the ladder to the first support using the climbing mechanism included in the tower base section. In picture 3 a third support is added, the climbing ladder is moved up to a second climbing frame and the crane is ready for climbing. In the fourth picture the crane is in position and the lower frame can be removed.

The upper support transfers only horizontal forces to the building, while the lower support transfers vertical and horizontal forces. The maximum free standing height of the crane above the upper support as measured from the bottom of the jib is 70 ft 2 in for crane no. 1 (figure 2.30) and 81 ft for crane no. 2 (figure 2.31).

Crane no. 1 was supported by the climbing frame, the lower support, on the 20th floor as shown in figure 2.33. The upper support was provided on the 22nd floor as shown in figure 2.34. This total distance is 18 ft, slightly less than the manufacturer's specified 18 ft 4 in. A typical corner clamp detail on the 22nd floor is shown in figure 2.35. Note the cracked slab where the clamp is attached.

Crane no. 2 was reported to be on either the 14th or 17th floor with most reports indicating the 17th. Reported conditions of shoring under both cranes varied from no shores to shores all the way to the foundation mat. Crane no. 1 has been positively located with its base support on the 20th floor. Shoring under that crane is less certain. OSHA investigators have indicated that it was not shored to the foundation until after the collapse.

In order to determine more accurately the location of crane no. 2 it is necessary to look at previous construction photos. Such photos were available from the OSHA file [18]. However, the exact dates of the photos are not known. It is known that construction records indicate that crane no. 1 was last raised on February 26 and crane no. 2 on February 7.

Figure 2.36 contains two construction photos reported as taken around February 10. This date is in error as will be shown. Since the lower floors are not visible it is not possible to tell from figure 2.36 where the cranes are located. Figure 2.37 shows a photograph taken on March 6 that can be used to establish the location of the 15th floor in figure 2.36. A certain pattern of "color" may be seen in the doors and door frames at the elevator landings. A comparison of the patterns between figures 2.36 and 2.37 (taken March 6) establishes the location of the 15th floor in figure 2.36 (It will be seen later that the same door pattern existed in a set of photos taken between figure 2.36 and 2.37).

Examination of figure 2.38 establishes the lower crane support on the 10th floor and establishes that the crane, as installed, could operate through a ten-story height of building. It should be noted that the crane had marginal clearance to cast the 20th floor although it appears that it could. The photos were not clear enough to determine shoring conditions under the crane.

Close examination of figure 2.38 shows openings in section 4 of the 20th floor forms. Therefore, the 20th floor had not been cast at the time of the photo. Reference to figure 2.3 indicates that this section was cast on February 1. Construction records indicate that framing for the 20th floor forms, which is visible in figure 2.38, was installed on January 27. Based on this evidence it appears that figures 2.36 and 2.38 were taken between January 27 and February 1. Therefore the photograph was taken prior to February 7 which was the last date crane no. 2 was raised.

Figures 2.39 through 2.42 were reported taken on February 11 or February 18, both dates after crane no. 2 was raised. The floor numbers can be established simply by counting at the east end of the building. Note the location of the 15th floor with respect to the top elevator landing. This location is in agreement with photos taken before and after figure 2.39. The elevator door pattern can be clearly seen in figure 2.40. Note also that at the time of the photograph, crane no. 1 is based on the 16th floor (crane no. 1 was raised on February 26) and crane no. 2 is based on the 14th floor. Since crane no. 2 could work through 10 stories (figure 2.38) it could have been used to cast section 3 of the 24th floor while based on the 14th floor.

The various tower sections of the crane are identified in figure 2.41. Note that five standard tower sections were used instead of the usual four (figure 2.31 and reference 22). Assuming the upper and lower supports were two stories apart as has been stated, the crane had a free standing height of about 99 ft which exceeds the maximum free height of 81 ft shown in figure 2.31. It is not known if any special precautions were taken to allow the difference between the actual and the recommended installation.

Examination of figures 2.40 and 2.42 indicates that either the crane shoring or the formwork for the concrete closure slab of the crane opening was in place in the third through thirteenth stories on the day of the photograph. Conditions below the third story are not visible in the photograph.

The date of the photograph (reported as February 11 or 18) can be checked by looking at figures 2.39, 2.42, and 2.2. Close examination indicates that none of the 22nd floor had been cast at the time of the photograph (openings can be seen in the slab forms). Since figure 2.2 indicates that section 1 on the 22nd floor was cast on February 14, it is established that the photograph was taken before that date. Due to the construction progress shown in the figure it can be stated that section 2 of the 21st floor which was cast February 6 (figure 2.2) was in place. Based on this evidence it appears that the photograph was taken between February 6 and February 14.

Although the forms are in place for sections 3 and 4 of the 21st floor, it cannot be positively stated that they have been cast. Therefore, on the basis of these photographs (figures 2.39 through 2.42) it cannot be conclusively stated that the crane which was raised on February 7 was based on the 14th floor.

However, based on figures 2.36 and 2.38 the crane no. 2 was attached to the 10th floor between January 26 and February 1. Since this crane was raised only once after February 1, it must have been located on the 14th floor at the time of the collapse.

It has been established by physical examination that crane no. 1 was based on the 20th floor. Examination of photographs and contractor's records have been used to conclude that crane no. 2 was based on the 14th floor.

Crane no. 2 was located in the failure zone. The position of the crane after the collapse will be discussed after consideration of the parking garage.

2.3 Parking Garage

The parking garage was a flat plate structure of post-tensioned unbonded concrete construction. The completed structure was to have four stories of parking with a landscaped roof. The B-4, or lowest level, was a slab-on-grade. The parking garage slabs were at approximately the same elevations as the corresponding floors in the building A-4 basement.

The plan view is shown in figure 2.1. A typical panel was 28 ft by 30 ft with an 8-in slab. The columns were supported on footings. The story height was 9 ft for stories B-4, B-3, B-2, and varied from 10 to 14 ft for story B-1. Normal weight aggregate concrete with a design compressive strength of 4,000 psi was specified throughout the structure.

Slabs B-4 and B-3 had been cast and slab B-2 had been cast to the extent shown in figures 2.1 and 1.2. Note that in the vicinity of grids lines BB to FF and grid line 1 slab B-3 was placed on compacted fill. Compacted fill was apparently placed around the three columns, BB-1, DD-1, and EE-1 in story B-4. The footings for these columns were a few feet higher than the rest of the column footings.

The parking garage, as seen from building A-4 is shown in figure 2.43. A number of columns have been identified for reference. At the time of the collapse, slab B-2 had been cast to the extent shown. The rest of B-2 had been formed (some of the forms had been cleared at the time of the photograph). A closeup of several of the columns in figure 2.43 are shown in figure 2.44.

The locations of slabs B-3 and B-2 are shown in figure 2.44 on column DD-4. Note the virtual lack of shear cones at the slab levels on most of the columns in that figure.

The garage slabs generally came "straight" down except near the slab edges. In the latter cases the columns usually failed in bending. Figure 2.45 shows a view of slab B-2 with the edge between columns JJ-5 and JJ-8 visible on the left. The view of this edge from the south is shown in figure 2.46. Note that the columns failed at the B-3 level. Formwork can be seen lying on top of the slab B-3. These forms apparently were in place under slab B-2.

Column KK-6 may be seen in figure 2.47. Once again the shear cone usually associated with a slab punching failure is not visible in the photograph. Note the ruptured reinforcing bars at the base of the column. These are shown more clearly in figure 2.48 (but not for column KK-6).

The east edge of the garage was supported by columns as shown in figure 2.49. The northern edge was simply supported by corbels on the columns. These corbels are visible in figures 2.49 and 2.50. The pool deck level (corresponding with the first floor of building A-4) was to rest on corbels on columns 25, 26, 27 and 33-37. Slab B-1 was to rest on corbels on columns 25, 26, 27, 29, 31-38, and shear wall H. Slab B-2 was supported by corbels on columns 25, 26, 27, 29, 31-38, and shear wall H. Slab B-3 was supported by corbels on columns 25, 26, 27, 29, 31-36, and shear wall H.

The loss of columns between shear wall H and column 33 meant the loss of two columns (31 and 32) which provided support for slabs B-2 and B-3. The total span between column 33 and shear wall H was about 65 ft.

The west edge of the garage was supported by bearing on a wall and framing into a ramp as shown in figure 2.51. Slab B-2 may be seen resting on the portion of B-3 cast on a slab-on-grade (vicinity of column EE-1). More detail of the ramp framing can be seen in figure 2.52. The ramp was of reinforced concrete construction. The west wall south of the ramp can be seen in figure 2.53 and with more detail in figure 2.54. The formwork and strand which had been positioned for slab B-2 can be seen in figure 2.53. A portion of a shear cone is visible on column KK-3 in the same figure.

The south edge of the garage was supported by columns which were independent of building A-5. This was in contrast to the corbel support system on the north edge.

The progress of formwork and stressing in the parking garage has not been established at the time of this writing. These factors must be determined prior to analysis of the garage since they have a direct bearing on the behavior of the structure.

This unbonded post-tensioned concrete construction such as used in the parking garage, is a common form of construction. However, consideration of its failure was beyond the scope of this report.

2.4 Crane No. 2

At the time of the collapse, crane No. 2 was reported idle with the jib pointed towards the garage. The exact orientation is not known. A distant view of the tower sections of the crane after the collapse may be seen in figures 2.49 and 2.51 (also 2.5). The tower top and hoist machinery (see figure 2.29) may be seen near column DD-2 in figure 2.55 (Note the partial shear cones on column DD-2). Based on an examination of the photographs it appears (refer to figure 2.1) that the crane tower lines up with the original position of the crane base (in plan view) and passes almost over columns 32 and DD-3, ending up with the top as shown in figures 2.55 and 2.56. This is a total distance of 125 to 130 ft. This distance compares to a total tower height (base section, 5 standard sections, and 1 slipring section) of about 125 ft. The implication here is that the tower fell while rotating about its original plan position. However, careful inspection of figures 2.54, 2.55, and 2.5 shows the jib is lying alongside the tower sections. The sequence of falling which would permit a jib initially pointing over the garage (as reported by employees) to assume its final position has not been determined.

3. CONCRETE AND REINFORCEMENT IN BUILDING A-4

3.1 Concrete

3.1.1 Standard Cylinder Tests

The standard (ASTM C39) cylinder test results for compressive strengths available at the time of this writing, are shown in figure 3.1 for the 17th and 21st through 24th floors [18]. Both 7-day and 28-day results are shown for section 3, the failure area. The results for section 3 indicate concrete meeting the strength requirements called for by the structural engineer (3000 psi for the slabs and columns). The results of standard cylinder tests, however, do not reflect the effect of field curing conditions or the strength of the concrete in place. Field-cured cylinders were not made as called for by OSHA regulations (ANSI A10.9, section 6.4.7) [2]. It should also be noted that the test results for the 22nd floor, section 2, and the 23rd floor, section 1, had strengths less than those normally obtained for other floors at 7 days. Both of these sections were outside the failure zone.

3.1.2 Field Cores

In order to evaluate the strength of the concrete in the building the portion of the slabs in section 3 between grid lines H and I (figure 2.1) were cored. Four-inch diameter cores were obtained from the structure in the 17th and 21st through 23rd floors. The results of compressive and split cylinder tests on the cores are shown in figure 3.2 [17]. The 17th floor was sampled because crane no. 2 was originally thought to be supported on this floor, although in section 2.2.3 it was concluded that the tower was based on the 14th floor. The other floors were sampled because of their proximity to where the failure started.

3.1.3 Estimates of Quality of Concrete as Delivered

The concrete quality as indicated by the standard 6-in diameter cylinders was satisfactory for the 22nd and 23rd floors in section 3 (figure 3.1). The field cores were used to check on the quality of concrete as delivered. This comparison is made by converting the average values of 4-in diameter core strength to 6-in standard cylinder values.

There has been conflicting information on whether strengths derived from cores are greater or lower than those from standard cylinders [3, 4, 8, and 16]. The strength of cores obtained from mass concrete (such as dams) frequently are greater than strengths derived from standard cylinders. However, Bloem [4], using 6-in thick slabs with normal weight aggregate concrete, has shown that 4-in diameter cores, such as obtained from building A-4, have lower strengths than standard 6-in diameter cylinders. Earlier Campbell and Tobin [8] obtained similar results using lightweight aggregate concrete and 4-in cores obtained from 12-in thick slabs. The data from Bloem is reproduced in figure 3.3 to show the relationships obtained. Campbell and Tobin data are similar but are not shown because of insufficient data at early ages.

Bloem tested cores from both well-cured and poorly-cured slabs. His well-cured slab was sprayed with a curing compound as soon as the water sheen had disappeared. Later the slab was covered with wet burlap and a plastic sheet. The burlap was kept wet and in place for 14 days. At that time it was removed and the slab raised to permit air circulation. The poorly-cured slab was left uncovered after pouring. Three days later the forms were stripped and the slab was raised from the floor. The environmental conditions in the period between placement and coring of the slabs ranged in temperature from about 60 to 90° F and in relative humidity from about 25 to 90 percent. The field curing conditions for each floor in building A-4 are not known.

Estimates were obtained for a range of standard 6-in diameter cylinders by converting from the age at time of test to 28-day 6-in cylinder strength using the curves of figure 3.3. The calculations rounded to the nearest 5 psi follow:

17th floor: well-cured slab 2780/0.98 (fraction @ 66 days)
= 2835 psi

poorly-cured slab 2780/0.72 (fraction @ 66 days)
= 3860 psi

21st floor: well-cured slab 2470/0.90 (fraction @ 38 days)
= 2745 psi

poorly-cured slab 2470/0.72 (fraction @ 38 days)
= 3430 psi

22nd floor: well-cured slab 1960/0.82 (fraction @ 25 days)
= 2390 psi

poorly-cured slab 1960/0.68 (fraction @ 25 days)
= 2880 psi

23rd floor: well-cured slab 2295/0.73 (fraction @ 16 days)

= 3145 psi

poorly-cured slab 2295/0.62 (fraction @ 16 days)

= 3700 psi

Within the limitation of the scatter observed in the core test results (see figure 3.2), the above data, when compared to the standard 6-in cylinders (figure 3.1) indicate that the concrete quality as delivered was generally acceptable although concrete strength of the 22nd floor slab appears to be low. The foregoing is not a measure of the strength of in-situ concrete which is discussed in the next section.

3.1.4 Estimates of Concrete Strengths on the Day of Collapse

The many factors influencing concrete strength have been discussed elsewhere [3, 14, 16, and 23] and are not repeated here. However, it should be pointed out that for the same concrete:

(1) Strength will decrease with a deficiency in curing moisture after the initial set of the concrete.

(2) Strength potential will increase with sustained low temperatures (above freezing), although strength will be less at the usual test periods (7 and 28 days).

(3) Strength development will accelerate with high temperatures, although the eventual maximum strength will be less than in (2).

(4) Calcium chloride will accelerate the rate of strength development although it will not prevent freezing.

A set of curves have been drawn in figure 3.4 from data obtained by Klieger [14]. These curves, although obtained from normal weight aggregate concrete, indicate the effect of curing temperatures and 2 percent calcium chloride on concrete compressive strength development. The 73°F curing curve without calcium chloride represents strength gain under standard conditions. Note the accelerating effect on strength development of 2 percent calcium chloride. For example, a concrete cured at 55°F with 2 percent calcium chloride is equivalent in strength (during the first 28 days) to the same concrete cured at 73°F. Although not used in all floors, 2 percent calcium chloride was used for the 17th and 21st through 24th floor slabs.

The cores are used as a measure of in-situ strength. The Klieger data require the use of 28-day strengths. The core strengths for the 22nd and 23rd floors are converted to 28-day strengths by using figure 3.3. The basic core data are multiplied by the ratio of the 28-day fraction to the fraction at the date tested. These calculations rounded to the nearest 10 psi follow:

22nd floor:	well-cured slab	1960x0.85/0.82 (fraction @ 25 days)
		= 2030 psi
	poorly-cured slab	1960x0.69/0.68 (fraction @ 25 days)
		= 1990 psi
23rd floor:	well-cured slab	2295x0.85/0.73 (fraction @ 16 days)
		= 2670 psi
	poorly-cured slab	2295x0.69/0.62 (fraction @ 16 days)
		= 2550 psi

In order to convert these strengths into estimates at the time of collapse the curing temperature should be considered. The average temperature history has been shown in figure 3.5 (enlarged from figure 2.3). The average air temperature was 42°F for the 22nd floor and 45° for the 23rd floor (temperatures as recorded at National Airport). Estimates of strength using the Klieger data for 2 percent calcium chloride in figure 3.4 (and interpolating between temperature curves) yields the following results rounded to the nearest 10 psi:

22nd floor:	well-cured slab	2030x0.66 (fraction @ 10 days)
		= 1340 psi
	poorly-cured slab	1990x0.66 (fraction @ 10 days)
		= 1310 psi
23rd floor:	well-cured slab	2670x0.45 (fraction @ 4 days)
		= 1200 psi
	poorly-cured slab	2550x0.45 (fraction @ 4 days)
		= 1150 psi

In the prediction of concrete strength it is recognized that the average in-situ strength of the floor slabs in question may be different because of non-uniform curing temperatures, effects of coring and size of core, scatter in test results in seemingly identical core specimens (see figure 3.2), chilling factor of the wind and approximations introduced by application of test results from independent sources of study. Consideration of these factors, including quantitative assessment whenever possible, indicates a range in the order of $\pm$ 20 percent on the average concrete strength estimate.

The structural engineer's calculations for evaluation of shear strength for the structure requires the use of a splitting ratio [6] for the lightweight concrete. The structural engineer's design calculations (page 26A, floor slab design for building A-4) provided data indicating that a splitting ratio of 6.0 could be used. The splitting ratio is equal to the ratio of splitting tensile strength to the square root of the compressive cylinder strength (figure 3.2). The core data in figure 3.2 indicate a splitting factor of 6.5 to 7.0.

3.2 Reinforcing Steel

Examination of the reinforcing steel in the area bounded by columns 66-68-83-86, indicates that the steel called for by the structural engineer was included in the reinforcing steel shop drawings. The structural engineer generally called for no.4 bars in the slab. Slabs were not available for examination to check if steel was actually placed in accordance with shop drawings.

A total of 12 reinforcing bars were tested, 6 of these were removed from floor slabs of the collapsed section of the building and 6 were tested from unused reinforcement steel at the construction site. The results of these tests are shown in figure 3.6 [17].

The structural engineer generally called for reinforcing steel with a 60,000 psi yield point and meeting the requirements of ASTM A-615. The steel tested satisfies the requirements of that specification.

4. STRUCTURAL INVESTIGATION OF FAILURE CONDITIONS IN BUILDING A-4

4.1 Introduction

A structural investigation of the 22nd and 23rd floors of the Skyline Plaza apartment building A-4 in the region where the collapse occurred was conducted. The investigation consisted of:

(1) determination of internal forces in slab, beam, and column elements of the structural assembly in accordance with the principles of elastic analysis;

(2) evaluation of the ability of structural elements to resist previously determined forces.

The three-dimensional elastic analysis was performed using the finite element analysis program known by the acronym SAP [22A] and the computer facilities at the National Bureau of Standards. The simulated model of the structural assembly shown in figure 4.2 consists of three-dimensional beam elements simulating beams and columns of the system and quadrilateral plate bending elements simulating the floor slab. The beam element properties are discussed in the SAP user manual while the plate element properties are described in reference [9].

The evaluation of the capacity of structural elements was based on the provisions of the ACI 318-71 Code [7] and on procedures from published analytical and experimental research [12, 13, 13A, 15].

4.2 Controlling Regulations and Basis of Design

The basic national standard governing the design of reinforced concrete buildings is the Building Code Requirements for Reinforced Concrete by the American Concrete Institute, ACI 318 [6,7]. New editions of the ACI Code are issued periodically to provide for the advancement in the state of the art resulting from research and professional experience. The latest edition of ACI 318 was issued in 1971 [7] and the one prior to that, in 1963 [6].

In building design the applicable building code is the latest edition available at the time of the design. In the case of the Skyline Plaza apartment building, the design should comply with the requirements of the 1956 edition of Building Code Ordinance (amended in 1971) of Fairfax County, Virginia [5], which incorporates the design provisions of ACI 318-63.

It is noted that for certain areas of the floor the spacing of columns is such that in order to comply with section 2101(e)2 of ACI 318-63, the slab thickness would have to be greater than 8 in. Section 3 of the floor slab, as identified in figure 2.1 contains such areas.

4.3 Analytical Procedure

Whereas the design of the structure was governed by ACI 318-63, the analysis that follows is based upon ACI 318-71 which represents the present state of the art. The calculations of shearing stresses were made in accordance with the procedure described in section 11.13.2 of the Commentary to the ACI 318-71. The latter is essentially based on the results of experimental studies by Hanson and Hanson [13] of slabs supported by square columns. In addition, consideration was given to the effect of column rectangularity on shear capacity as reported by Hawkins, Fallsen and Hinojosa [13A]. For distribution of loads between shored slabs consideration was given to findings by Grundy and Kabaila [12], Nielsen [15], and observations of field measurements of high-rise flat slab buildings related by Agarwal and Gardner in an unpublished report.

Three finite element analyses were made for the slab assembly in the region of collapse. The approximate area of collapse is shown in figure 4.1. All three analyses were based on the grid model shown in figure 4.2.

The accuracy of a finite element analysis is generally improved as the number of elements is increased and as the elements become more square. In constructing the grid, a finer mesh was used in the region west of the crane tower opening, where failure was assumed to have been initiated. The boundaries of the rectangular grid are approximately defined by the column line 18-73-72-79-28 on the east and column line 11-63-64-90-91-35 on the west. This area encompasses the failure region and extends about one column line beyond its boundaries on each side.

The slab element thickness was 8 inches as called for in the contract drawings. The supporting column elements were rigidly attached to the slab elements at the top and were fixed at their base. The columns above the floor being analyzed were also rigidly attached to this slab at their base with the tops hinged. Columns designated by G and 32 in the contract drawings were each simulated by a rigid frame consisting of two columns and a stiff beam.

The grid layout in figure 4.2 is different from the slab in the contract drawings in certain aspects. The balcony slabs were assumed extending to the centerlines of the end columns. Likewise the boundary of the slab at north and south fascias between balconies was assumed extending to the centerlines of the flanking columns, eliminating, in effect, the stepped portion of the slab between a balcony and an adjacent fascia. Wherever possible, rectangular or square plate elements were used to construct the grid. In order to make column centroids coincident with nodal points of the grid a few trapezoidal plate elements were also introduced. By using a finer mesh one could develop a model which would more precisely represent the actual assembly. However, such a refinement is not justified because it would not significantly affect the results [7A].

Each of the three finite element analyses had a particular purpose:

<u>Case 1</u>: Some of the employee statements indicated that the 22nd story forms were entirely removed at the time of collapse (see section 2.2.2). This analysis of the 23rd floor system is based on the premise that

the 23rd floor slab was not shored. The loading consisted of the weight of the newly poured slab of the 24th floor (80 pounds per square foot), the weight of the formwork on the 23rd floor (5 pounds per square foot) and the weight of the 23rd floor (80 pounds per square foot). Thus, a total uniform load of 165 pounds per square foot was placed on the portion of the slab directly below the poured section of the 24th floor (section 3) and a load of 85 pounds per square foot was placed elsewhere. The modulus of elasticity used for the concrete slab of the 23rd floor was calculated in accordance with ACI 318-71 using the estimated average concrete strength of 1200 psi (see section 3.1.4), and an approximate density of 120 pounds per cubic foot (see table 3.2). A similar calculation was made for the elastic moduli of the columns. For stiffness calculations, the moment of inertia was assumed to be that of the gross section of the concrete.

Case 2: To check the influence of estimated concrete strength on the analytical results, Case 2 assumed the concrete to have attained its full 28-day design strength of 3000 psi. The 23rd-floor system was analyzed using elastic moduli calculated as in Case 1 except that design concrete strength was used.

Case 3: Some of the employee statements indicated that removal of shoring under the 23rd-floor slab in the central corridor area of the building was in progress on the day of the collapse (see section 2.2.2). A loading simulating the condition of partial removal of shoring was used. This Case is similar to Case 1 except the loads on the 23rd floor slab were reduced outside the region bounded by grid lines 7-320, 320-330, 330-16, and 16-7 as indicated in figure 4.2. To obtain these reduced loads, the loads from case 1 were multiplied by a factor which was based on the assumption that 22nd and 23rd floor slabs shared the loads on the 23rd floor in proportion to their respective elastic moduli. The modulus of elasticity of the 23rd floor slab was the same as was used in Case 1. The modulus of elasticity of the 22nd floor slab was calculated on the same basis as described in Case 1 using the average estimated concrete strength at the time of collapse of 1340 psi for the 22nd floor slab (see section 3.1.4).

4.4 Discussion of Results

Figures 4.3 through 4.5 show the analytical results for cases 1 through 3. Moments M_x (rotation about x-axis) and M_y (rotation about y-axis) are designated by horizontal and vertical lines, respectively. They are shown plotted in pairs (and approximately to scale) at the centroids of slab elements. The numerical entries are moment magnitudes at the centroids and are expressed in units of foot-kips per linear foot of slab. Moments producing compression in the top of the slab are positive.

Figures 4.6 through 4.9 show moment diagrams developed for column strips taken from the grid of figure 4.2. These strips were investigated for their adequacy to resist moments obtained by analysis. Negative moments were investigated at the face of the columns according to ACI 318-71.

The slab was also investigated at column locations for its adequacy in shear. Shearing stresses were determined following the procedure described in section 11.13.2 of the commentary to the ACI 318-71 code. The applicable equations are given below for convenience. Figure 4.10 shows the critical section for shear around the periphery of a rectangular column and other parameters used in the following equations:

$$v_{max} = \frac{V}{A} + \frac{a_y M_y}{J_y}\left(\frac{h+d}{2}\right) + \frac{a_x M_x}{J_x}\left(\frac{b+d}{2}\right) \quad . \ . \ . \ . \ . \quad (4.1)$$

$$J_x = \frac{d(b+d)^3}{6} + \frac{d^3(b+d)}{6} + \frac{d(h+d)(b+d)^2}{2}$$

$$J_y = \frac{d(h+d)^3}{6} + \frac{d^3(h+d)}{6} + \frac{d(b+d)(h+d)^2}{2}$$

$$a_x = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\frac{b+d}{h+d}}}$$

$$a_y = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\frac{h+d}{b+d}}}$$

$$A = 2d\ (b+h+2d)$$

where

v_{max} = maximum shear stress in the slab

M_x = bending moment about x-axis transmitted to column

M_y = bending moment about y-axis transmitted to column

b,h = sectional dimensions of rectangular column

d = distance from centroid of tensile reinforcement to bottom face of slab

a_x, a_y = moment reduction factors defined above

J_x, J_y = properties of slab section as defined above

V = vertical load transmitted to column

A = area of critical slab section peripheral to column as defined above

According to ACI 318-71, the limiting value for shear stress is $4\sqrt{f'_c}$ for normal weight concrete (section 11.13.2) and 85 percent of that value or $3.4\sqrt{f'_c}$ for "sand-lightweight" concrete (11.3.2) such as used in the floor slabs. Critical shear in the slab will occur when the maximum shear stress determined by equation (4.1) exceeds $3.4\sqrt{f'_c}$ or,

$$v_{max} > 3.4\sqrt{f'_c} \quad \text{. . . . . (4.2)}$$

Alternately, expression (4.2) may be solved to give the minimum concrete strength required to prevent shear failure,

$$f'_c = 0.086 \; v^2_{max} \quad \text{. . . . . (4.3)}$$

Therefore, the shear capacity of the slab is considered to be exceeded when the required compressive strength determined in accordance with equation (4.3) is greater than the estimated strength at the time of collapse.

In addition to flexure or shear type failure in the slab, the collapse could have been attributed to other causes such as excessive creep in concrete or failure of one or more of the supporting columns. A limited investigation indicated that the columns had sufficient capacity to resist the applied loads. The possibility of creep being a significant contributory factor in the collapse was ruled out because of the short duration of peak loads on the 23rd and 22nd floor slabs prior to the collapse (about 4 hours) and the presence of reinforcement in the compressive regions of the slabs.

The scope of the following investigation is based on the hypothesis that failure conditions were governed either by flexure or by shear.

A. Investigation of Flexural Failure

A comparison of results shown in figures 4.3 and 4.4 reveals the internal moments in the slab from the analyses of Cases 1 and 2 to be virtually identical. This indicates that the accuracy of predicted concrete strength will not have a significant influence on the results of flexural investigations.

Altogether four column strips were investigated. Of these, the strip shown in figure 4.6 was found to be the one most critical in flexure. Case 1 negative moment at the critical section near column 31 (node 258) was about 9.2 ft-kips/ft. The moment capacity of the slab at this section, determined on the basis of reinforcement details in the contract drawings, was found to be about 8.4 ft-kips/ft, indicating that the flexural capacity of the section was exceeded by about 10 percent. In this case, as in all other cases investigated, the flexural capacity of the section was found to be governed by yielding of the tensile reinforcement rather than by crushing of concrete in compression. Thus it is estimated that yielding occurred in the slab at column 31 and yielding propagated toward column 83 (see figure 4.1). However, this local yielding is not sufficient to cause a collapse mechanism of the 23rd story floor slab. In fact, a flat slab construction of the type used in Skyline Plaza building is generally recognized to have considerably greater moment capacity than the capacity at which the first local yielding occurs because of its inherent ability to redistribute peak moments to neighboring regions through the mechanism of yielding.

The next most severe condition occurred in the column strip shown in figure 4.9. Maximum negative moments in elements adjacent to column 84 (node 165) were in the order of 7 to 8 ft-kip/ft at the critical section. This compares with a 10 ft-kip/ft ultimate moment capacity in the slab. The difference is not large enough to preclude yielding or even some yield line propagation from column 84 in the east-west direction; however, if such yielding were to occur the situation would be very similar to the localized condition around column 31. Again, it is concluded that this was not a significant contributory factor to the collapse of the 23rd floor.

To further check the ultimate flexural capacity of the floor slab a selected critical area was investigated using yield line analysis. The yield line pattern assumed for the balcony panel bounded by columns 32, 84, 83 and 31 is shown in figure 4.11. Top and bottom reinforcement taken from the contract documents is indicated in the same figure. This particular analysis gave an ultimate load capacity of about 300 pounds per square foot indicating 80 percent excess capacity over construction loads. Other yield line analyses performed at selected interior panels gave consistently similar results. It should be noted that unless the assumed yield line pattern happens to be the same as would actually develop in the slab, the approach used will overestimate the slab capacity. However, the margin of approximation involved would be much less than the excess capacity indicated by these analyses.

The possibility that the collapse might be attributed to flexural failure in the 22nd story slab was also considered. The most critical situation would occur when the 23rd story slab is fully shored and no reshores are installed under the 22nd floor slab (some employee statements indicate that the 23rd floor slab could have been fully shored, as discussed in section 2.2.2). This condition will be designated hereafter as Case 4. In this case, the loads acting upon the 22nd floor would consist of its own weight, the weight of of the 22nd story shoring and the loads transmitted from the

23rd floor. Assuming the loads on the 23rd floor to be distributed to both slabs in proportion to their respective elastic moduli, and shoring to be perfectly rigid [12], the maximum total load that could act on the 22nd floor would be about 4 percent greater than those assumed acting on the unshored 23rd floor (Case 1). Since both slabs are of identical design, the internal forces (shears, moments and torques) in the 22nd floor slab would likewise be about 4 percent greater. However, the flexibility of timber shores would modify the load distribution between slabs significantly [15], with the net effect of reducing the loads transmitted to the 22nd floor (viz.: a 100 percent flexible shore transmits no loads). Further reduction in transmitted loads would occur as a result of poor shoring conditions which appears to be a reasonable assumption to make (see section 2.2.2). Taking all these factors into account, it is concluded that flexure in the 22nd floor slab did not constitute the initial mode of failure. This conclusion is further corroborated by observations made elsewhere in this report (see section 2.2.2 and figure 2.28). It should be noted that since the 23rd floor slab, when fully shored, is less critical in flexure than either in Case 1 (no shoring) or in Case 3 (partial shoring), it requires no consideration with respect to Case 4.

Upon examination of all the probable conditions prior to the collapse, it is concluded that the initial mode of failure of either the 22nd or the 23rd floor slab was not flexural.

B. Investigation of Shear Failure

From the analytical results, axial loads and moments transmitted to column elements from the floor system were assessed in relation to peripheral column dimensions in order to establish which of the columns would be expected to create a severe shear condition in the slab. Figure 4.12 shows the four most critical regions identified at columns 67, 68, 83 and 84.

Figure 4.13 tabulates the results of the analysis of shear stresses in the 23rd-story floor slab for cases 1 through 3, based on the critical sections shown in figure 4.12. Maximum shear stresses calculated in accordance with eq. (4.1) are given in column 3. Column 4 gives the required compressive strength to resist this shear as determined from equation (4.3) and column 5 gives the probable range of in-situ concrete strength at the time of collapse.

According to recent studies [13A] the maximum shear capacity of a slab is significantly reduced when the supporting column has a narrow rectangular cross section as compared to a square column. In the case of column 68, the reduction in strength would be in the order of 20 percent. The bracketed values in column 4 of the table reflect the effect of column rectangularity.

Examination of the tabulated results reveals that the shear capacity of the 23rd floor slab is exceeded at columns 67, 68, 83, and 84 under unshored and partially shored conditions (Cases 1 and 3, respectively). It can be seen by comparing Case 1 values in column 4 with the probable range of compressive strength in column 5 that each of the four regions of the slab at the columns is critical in shear. For Case 3, conditions are critical at columns 84 and 68 and marginal at columns 83 and 67. Shear stresses at these columns are comparable to stresses in Case 1, indicating that partial removal of shoring creates conditions nearly as critical as would develop in the unshored slab. Note that none of the strength requirements exceed the 3000 psi design strength of the concrete considered in Case 2. The loss of support as a result of shear failure at any one of these columns would cause a progressive propagation of failure at neighboring columns and eventual rupture of the slab.

The cases discussed above represent conditions under which the 23rd story slab is the one most critically stressed in shear. The possibility of shear failure having first been initiated in the 22nd story slab was also examined. The most critical condition in this slab would occur under Case 4 loading (23rd floor fully shored and 22nd floor not reshored).

As previously noted, with infinitely rigid shoring, the load acting on the 22nd floor in Case 4 would be about 4 percent greater than the Case 3 load acting on the 23rd floor. However, the actual load distribution between the two floors would be significantly modified by the flexibility of the timber shoring [15] with the net effect of reducing the load transmitted to the 22nd floor. The load transmitted would be decreased further by the poor condition of shoring such as described in section 2.2.2.

The effect of these two factors on the load distribution cannot be quantified because of insufficient information. However, the load distribution which would simultaneously mobilize both floors towards their respective shear capacities can be examined; a condition under which the combined resistance of both floors is a maximum. In this case, the total weight of three floors plus shoring in two stories would be distributed between the 22nd and 23rd floors in proportion to the square roots of their respective compressive strengths. The results indicated required compressive strengths of 1030 psi and 1115 psi for the 23rd and 22nd floors, respectively. These compare with 1200 psi and 1340 psi respective average values estimated for these floors. Consequently, failure under Case 4 loading cannot be precluded. However, the following evidence is taken as an indication that the collapse did not initiate in the 22nd floor slab: (a) employee statements did not indicate sagging of the 22nd floor slab at the time of collapse and (b) one employee statement indicating the loss of the 21st story reshores prior to the incident was interpreted as a decrease in the 22nd floor deflection (section 2.2.2 and figure 2.28).

The contract documents (structural drawings) specify the following: "Slab being poured to be shored for two floors and backpropped at center of span each way and at center of bay on next floor down." Uncertainty about the effectiveness of backpropping or its presence prior to the collapse (section 2.2.2 and figure 2.27) makes it virtually impossible to make a quantitative assessment of loading distribution between 23rd through 21st floor slabs for this shoring configuration.

Upon examination of all the probable conditions prior to the collapse, it is concluded that the initial shear mode of failure of the 23rd story floor slab resulting from partial or complete removal of shoring prior to the incident was a major contributing factor to the collapse.

4.5 Probable Mode of Collapse in Building A-4

The most likely mode of collapse has been determined to be a shear failure around columns 67, 68, 83, or 84. The premature removal of forms supporting the 23rd story slab when the concrete of that slab had a relatively low strength produced shear stresses that were in excess of the concrete capacity at the time of the incident.

The three-dimensional finite element analyses have shown the slab to be overstressed in flexure in only a few local regions. The capacity of flat slabs to redistribute moments is well known and thus local flexural yielding should not have led to failure. Approximate ultimate flexural capacities were computed by the yield line analysis method. These ultimate flexural loads indicated that, even with the forms removed, the 23rd floor slab should not fail in flexure, thus confirming the interpretation of the results of the elastic analyses.

Hawkins test results [13A] indicate that shear cracking can cause concentrated slab rotations near the column that are quite large. Based on this observation it is felt quite possible that large deflections could have occurred, even with a shear type of failure that is estimated to have occurred. Most of the eyewitness reports indicated deflection in the 23rd and 24th story slabs (varying from 6 in to 2 ft) which increased over a 15 or 20 minute time period before failure. An increasing deflection of this type is usually associated with a flexural failure; however, this type of deflection could also be associated with a shear failure to the layman observer.

The collapse is believed to have started with shear around columns 67, 68, 83, or 84. The loss of support from any one of these columns would then lead to overstressing at the remaining columns.

The accumulation and impact of debris from the 23rd and 24th floor slabs would have overloaded the 22nd floor slab and induced progressive collapse of successive floors to the ground.

There is no indication that the crane was a contributing factor to the beginning of collapse in building A-4. No witness statements indicated that the crane moved prior to or during the initial sagging of the 23rd and 24th floors. The crane supports on the 14th and 16th floors are far enough away from the initiation of failure to preclude the crane as a cause. However, the crane probably became a driving force in the collapse once its support was lost.

5. SUMMARY OF FINDINGS

The findings given in this section are based on site investigations, OSHA case records, structural and architectural drawings, shop drawings, and structural calculations. The applicable Federal regulations are the Safety and Health Regulations for Construction [10] which incorporates the American National Standard A 10.9, Safety Requirements for Concrete Construction and Masonry Work [2].

5.1 Mode of Failure

On the basis of evidence as well as analysis, it appears that the collapse was initiated at the 23rd floor level.

An analysis of the 23rd-floor slab indicates that its most likely mode of failure was in shear around one or more columns in section 3 of the floor slab. The strength of the 23rd-floor slab on the day of collapse has been found to be of a magnitude that complete or partial removal of shoring underneath the slab would have produced a shear failure in the slab. The weight of debris resulted in failure in the slabs below and carried through the height of the building.

5.2 Non-Compliance with OSHA Regulations

Non-compliance with OSHA regulations was found in a number of instances. These are listed below along with a discussion of each item.

(a) Shoring in section 3 of the 22nd story

Examination of physical evidence and employees' statements indicate that the 22nd story forms were being removed on the day of the incident. OSHA regulations (ANSI-A10.9, section 6.4.7) require adherence to engineer's specifications and local building codes in determining length of time for forms to remain in place. The engineer's requirements were expressed in the form of a note on the structural drawings (section 2.2.2). This note required the "slab being poured to be shored for two floors and backpropped at center of span each way and at center of bay on next floor down." The architect's specifications [19] required "In all cases, two floors shall be fully shored." The removal of the 23rd story forms left only one story of formwork in place under the recently poured 24th floor.

(b) Premature removal of 22nd-story forms

The length of time forms were required to be left in place was not explicitly stated by the engineer, architect or local code. In such instances OSHA regulations (ANSI-A10.9, section 6.4.8) provide minimum curing times. When the design live load is less than the dead load, 4 days are required for spans less than 10 ft, 7 days for 10 ft to 20 ft spans, and 10 days for spans exceeding 20 ft. The time periods are for cumulative numbers of days in which the air temperature surrounding the concrete exceeds 50°F. The 4-day old 23rd floor slab had spans exceeding 10 ft. The forms removed on the 22nd story were in an area with spans exceeding 20 ft and therefore, should have been in place for 10 days of temperatures exceeding 50°F.

(c) Field-cured concrete specimens

OSHA regulations (ANSI-A10.9, section 6.4.7) require the use of field-cured concrete specimens in order to insure that concrete has obtained sufficient strength to safely support the load prior to removal of forms. No evidence has been found which indicates that field-cured specimens were prepared or used.

(d) Lateral bracing

OSHA regulations (ANSI-A10.9, section 6.3.2 and 8.1.5) require the design of braces and shores to resist all foreseeable lateral loads. Minimum values of 100 pounds per foot of floor edge or 2 percent of the total dead load of the floor, whichever is greater, is required. No evidence has been found which indicates that lateral load was considered in the design of forms. The lateral bracing provided (about 2 nominal 3x4's per 16 ft) would not provide this resistance.

(e) Shoring out of plumb

OSHA regulations (ANSI-A10.9, section 8.1.24) allow a maximum deviation of 1/8 in per 3 ft out of plumb. Deviations of shoring exceeding these limits were found on the 23rd (figure 2.20) and 24th (figure 2.16) stories.

(f) Damaged shoring

OSHA regulations (ANSI-A10.9, section 8.1.25) require removal of damaged or weakened shoring. On-site inspection after the incident indicates this was not done on the 23rd (figure 2.22) and 24th (figure 2.18) stories.

(g) Inspection

OSHA regulations (29 CFR 1926.700 (e) (1) (iv)) require inspection immediately before, during, and after placing concrete. This either was not done or deficiencies in (e) and (f) above were not corrected.

(h) Crane Installation

OSHA regulations (29 CFR 1926.550 (a) (1)) require the operation of cranes as prescribed by the manufacturer. The following deviations were found:

(1) Crane no. 1 (in section 1, away from the collapse area) - distance between top and bottom supports was less than the required 18 ft 4 in (figure 2.30) (18 ft was used).

(2) Crane no. 2 (in section 4, in the collapse area)- distance between top and bottom supports was less than the required 21 ft 4 in (18 ft was used).

(3) Crane no. 2 (in section 4, in the collapse area) - The number of standard tower sections used was one more than the four recommended by the manufacturer (figure 2.41 and section 2.2.3).

(4) Crane no. 2 (in section 4, in the collapse area) - The maximum tower height exceeded (by approximately one standard tower section) the 81 ft-0 in recommended (figure 2.31 and section 2.2.3).

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Figure 1.1 Appearance of completed apartment buildings.

Figure 1.2 Aerial view of construction at 11:00 a.m., March 2, 1973 (north is to the top).

Figure 1.3 Appearance of the collapsed structures as viewed from the southeast.

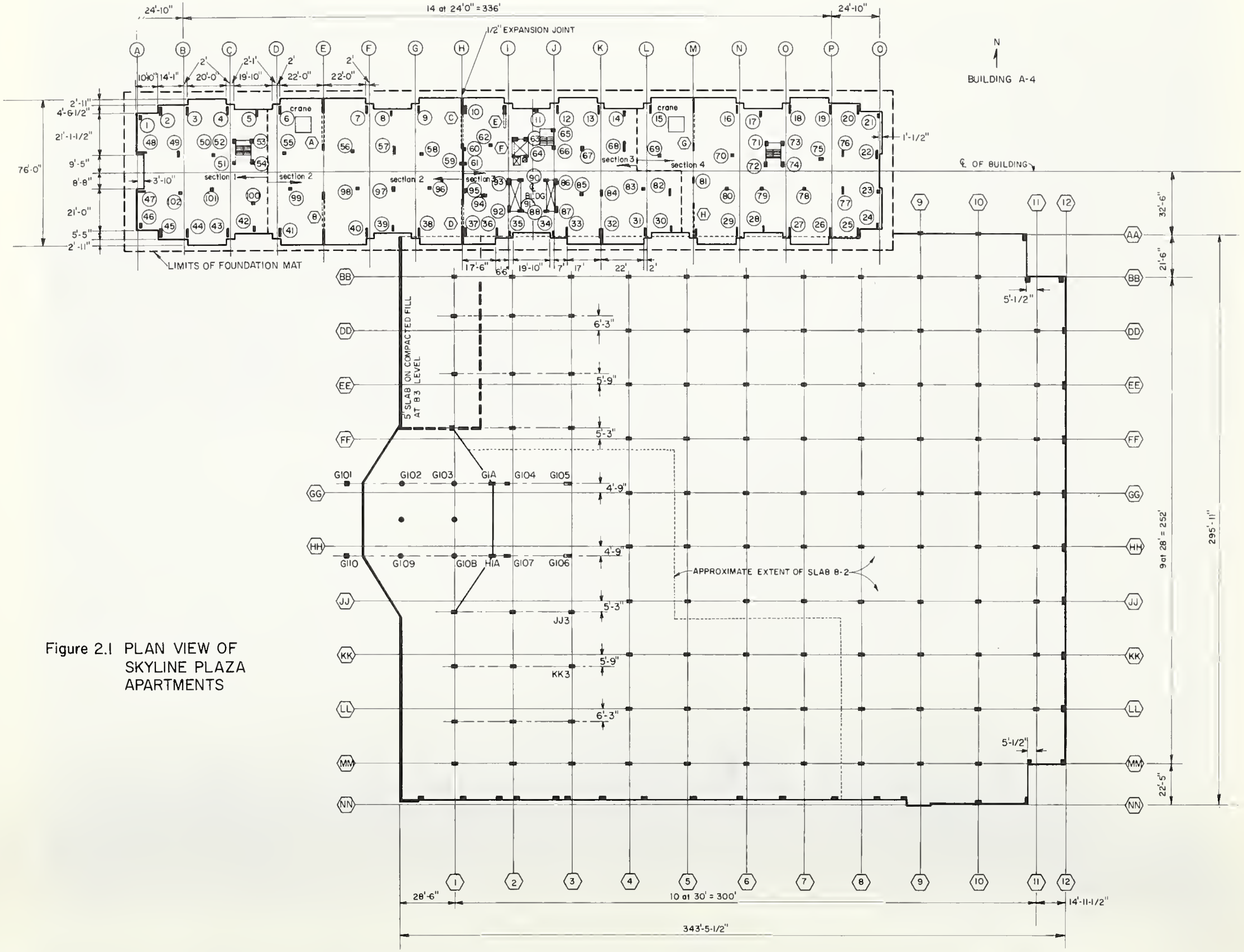


Figure 2.1 PLAN VIEW OF SKYLINE PLAZA APARTMENTS

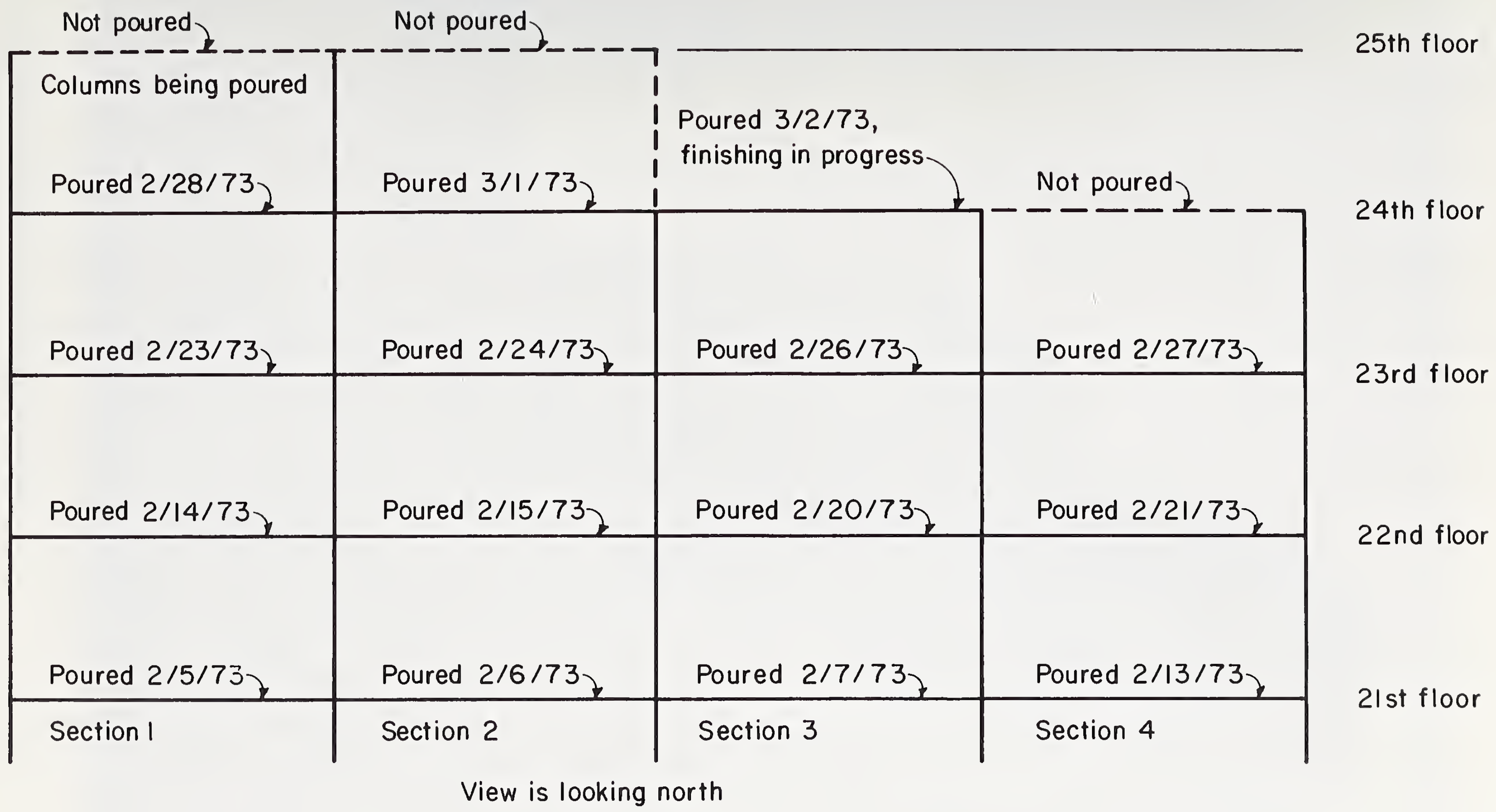


Figure 2.2 EXTENT OF CONSTRUCTION AT THE TIME OF COLLAPSE

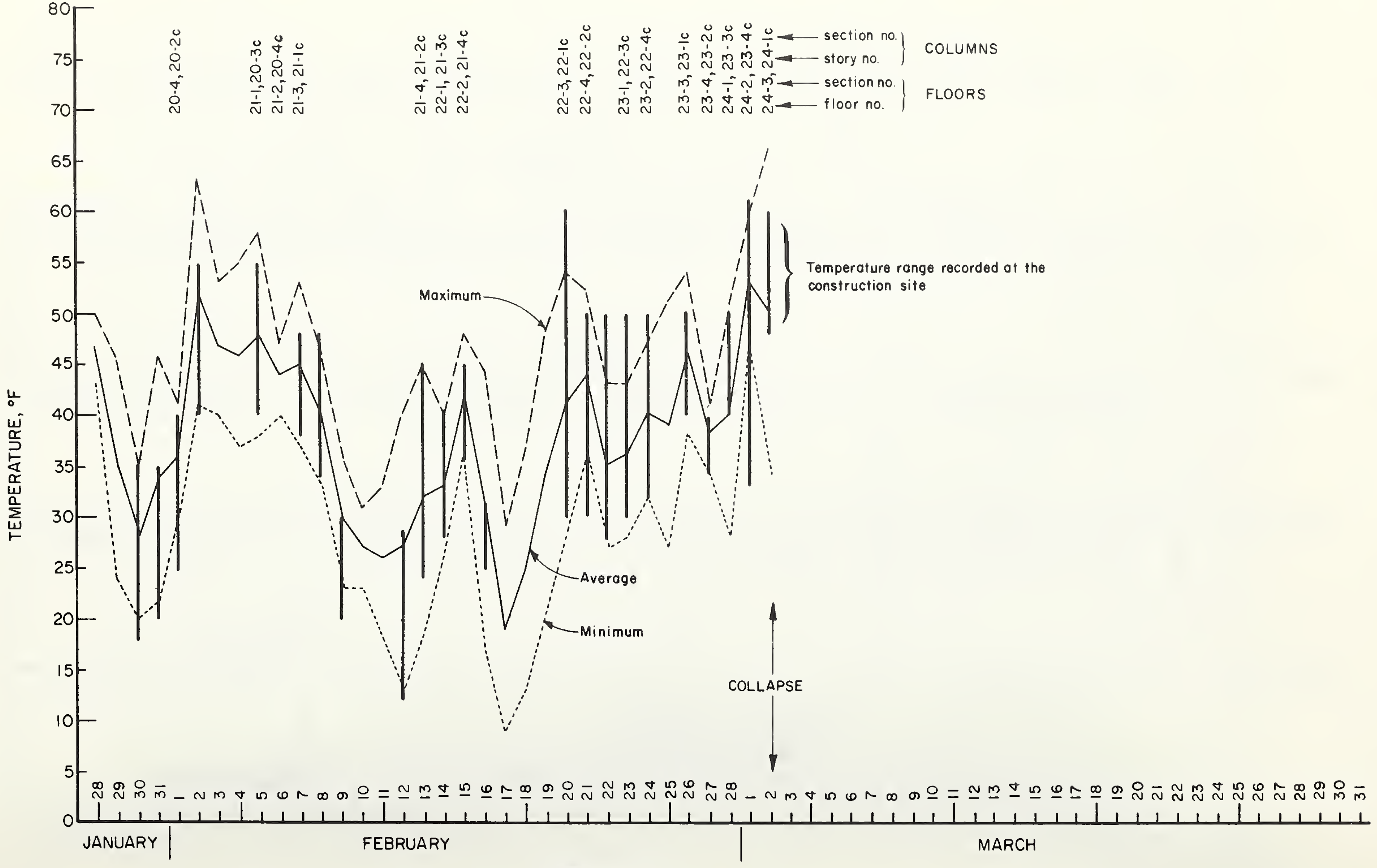


Figure 2.3 TEMPERATURE AND CASTING HISTORY FOR 21st STORY AND UP

Figure 2.4 General appearance of building A-4 as viewed from the southeast.

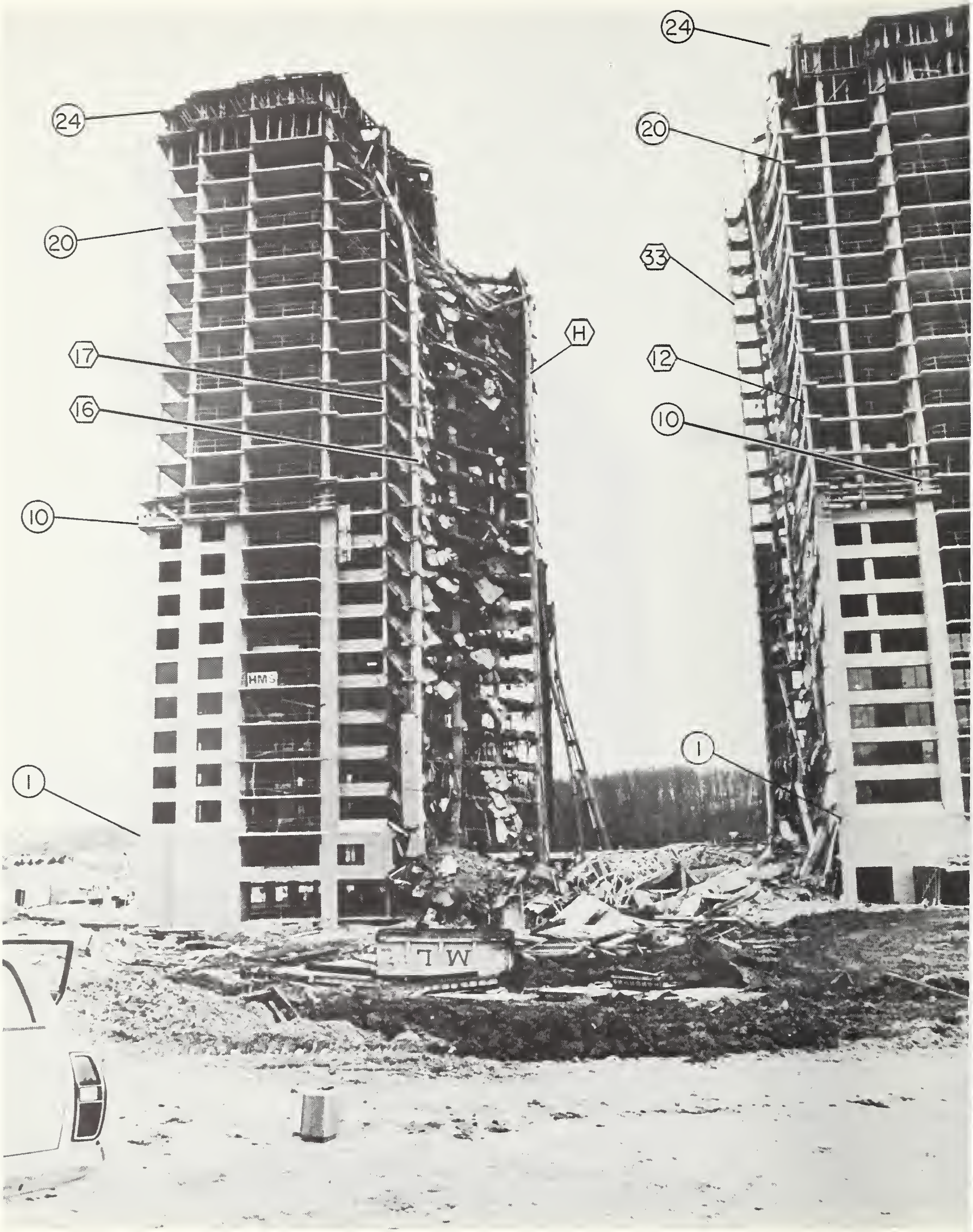


Figure 2.5 General view of the north face of building A44.

Figure 2.6 Closeup view of the top of the east end of the failure zone, looking south.

Figure 2.7 Closeup view near the top of the east end of the failure zone, looking south.

Figure 2.8 Closeup view near the midheight of the east end of the failure zone looking south.

Figure 2.9 General view of building A-4 as seen looking southwest.

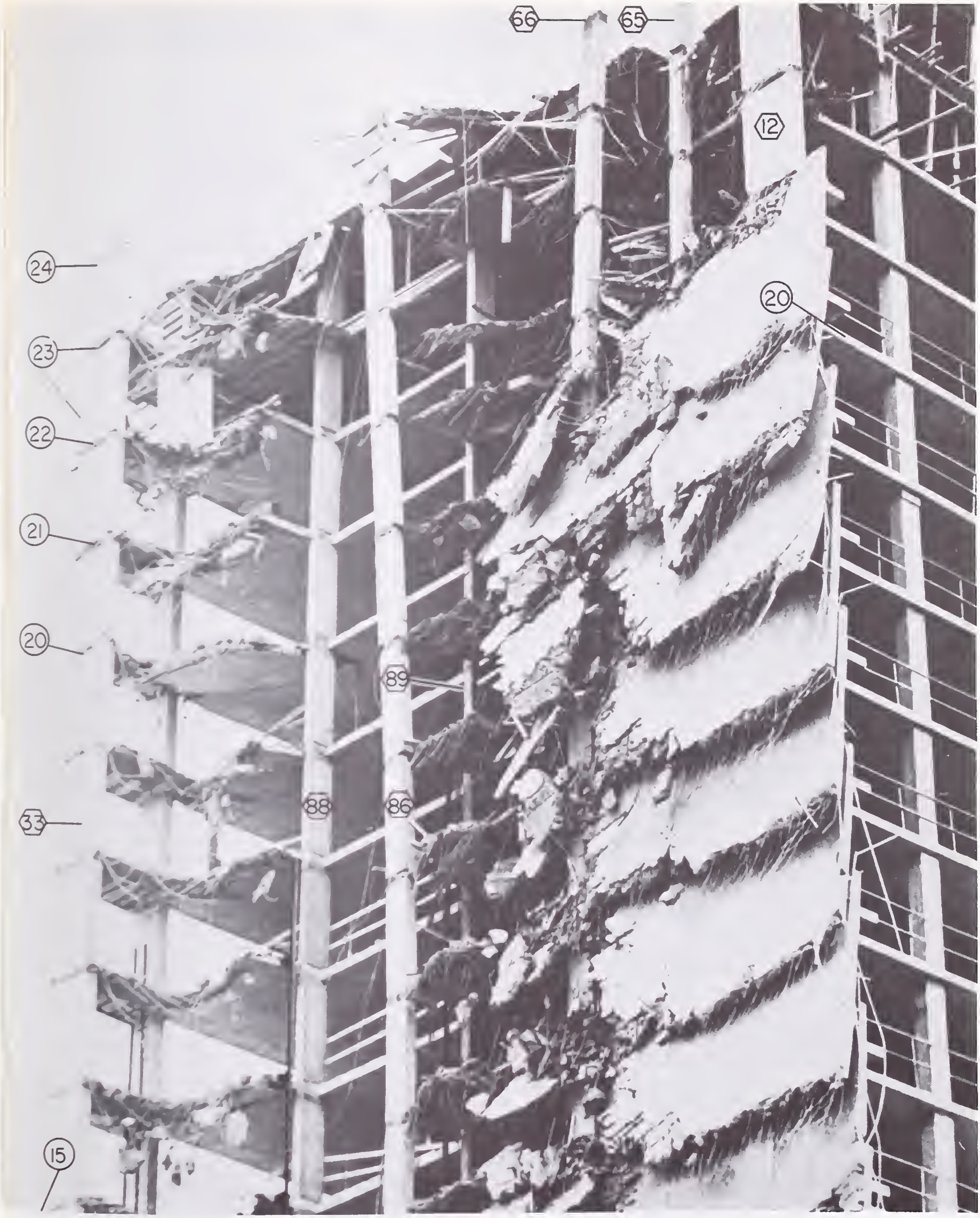


Figure 2.10 Closeup view of the top of the west end of the failure zone, looking southwest.

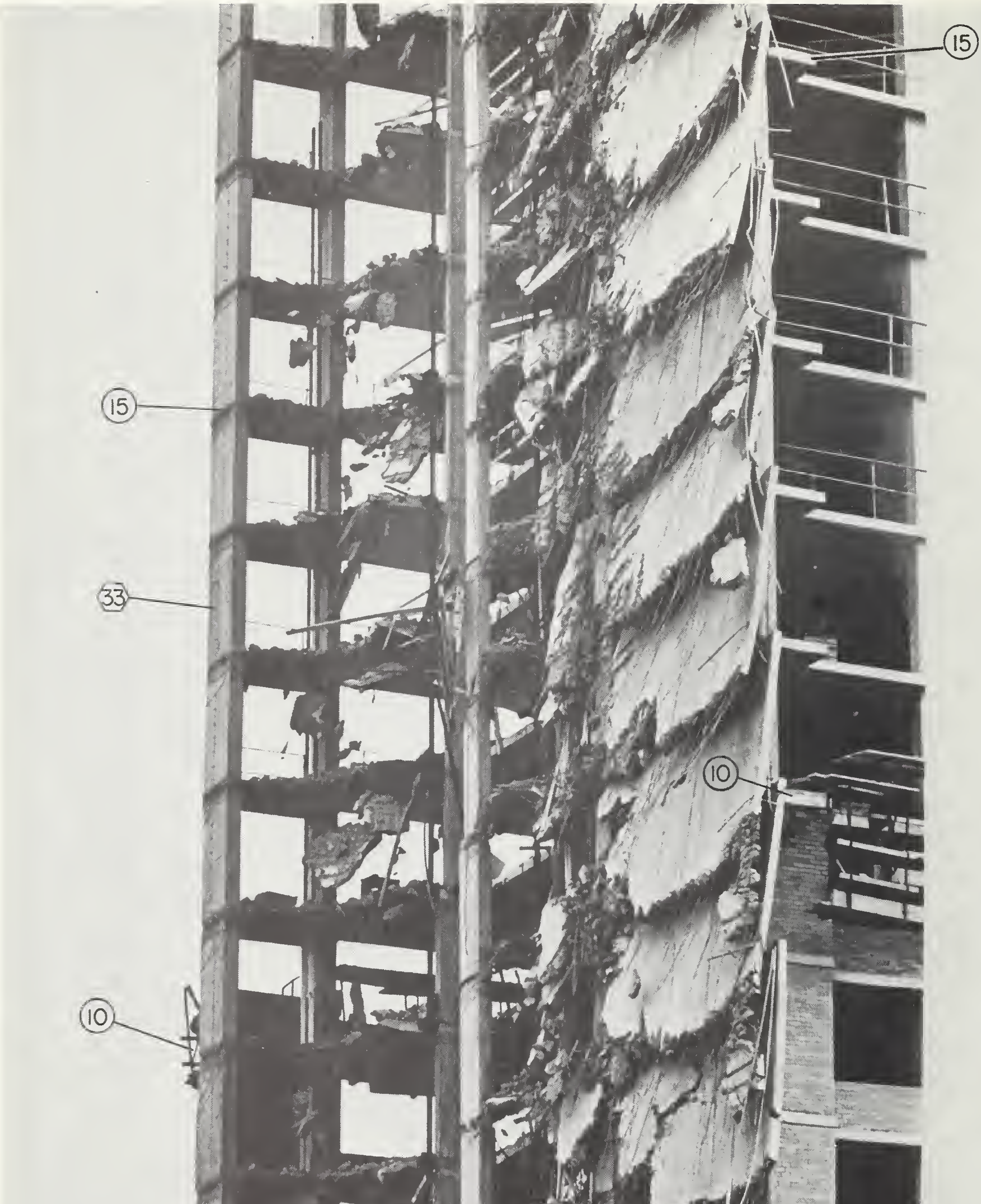


Figure 2.11 Closeup view near the midheight of the west end of the failure zone, looking southwest.

Figure 2.12 Closeup view at the bottom of the west end of the failure zone, looking southwest.

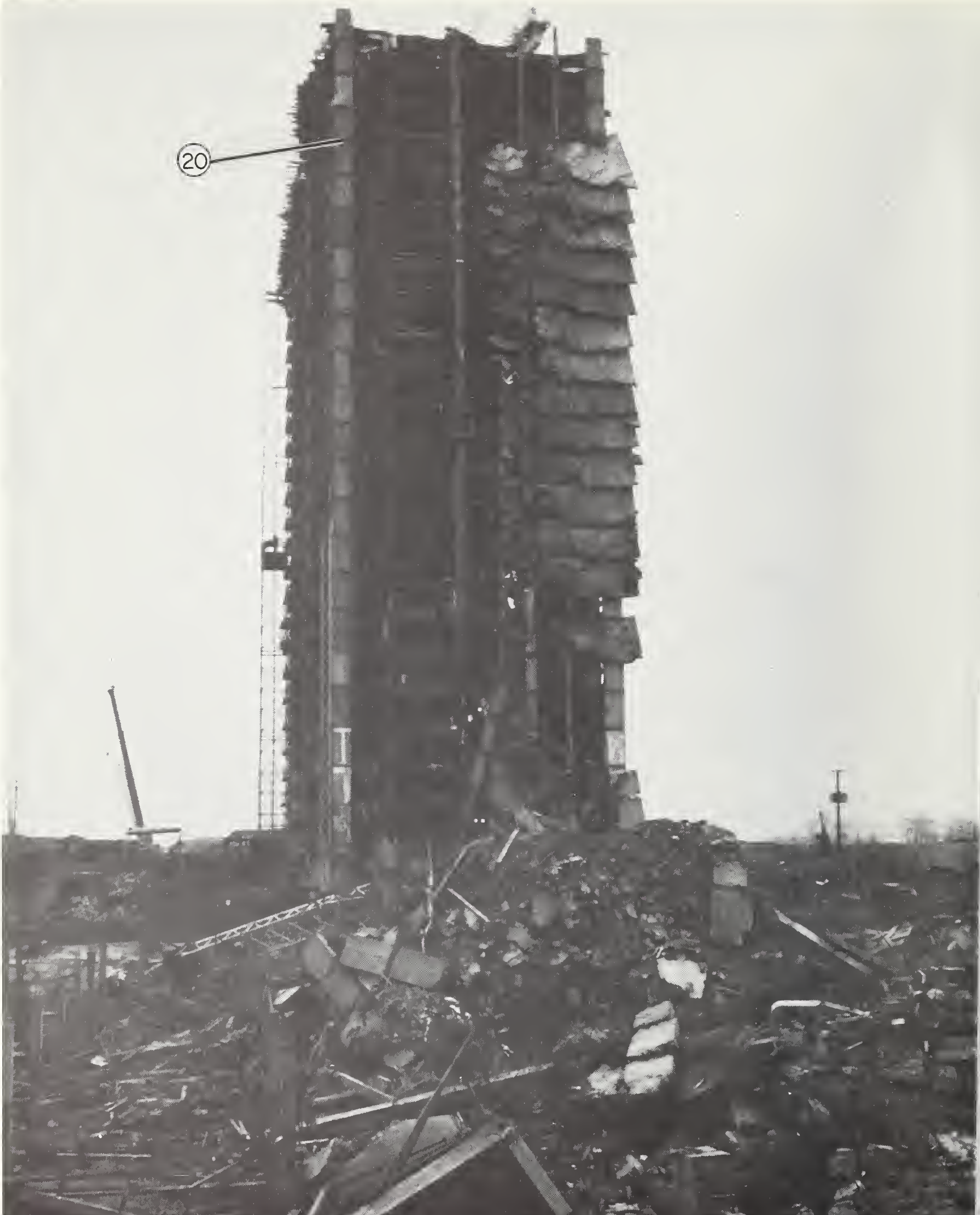


Figure 2.13 Appearance of A-4 after the removal of the east end of the building, viewed from the east.

Figure 2.14 North face of building A-4 after partial demolition.

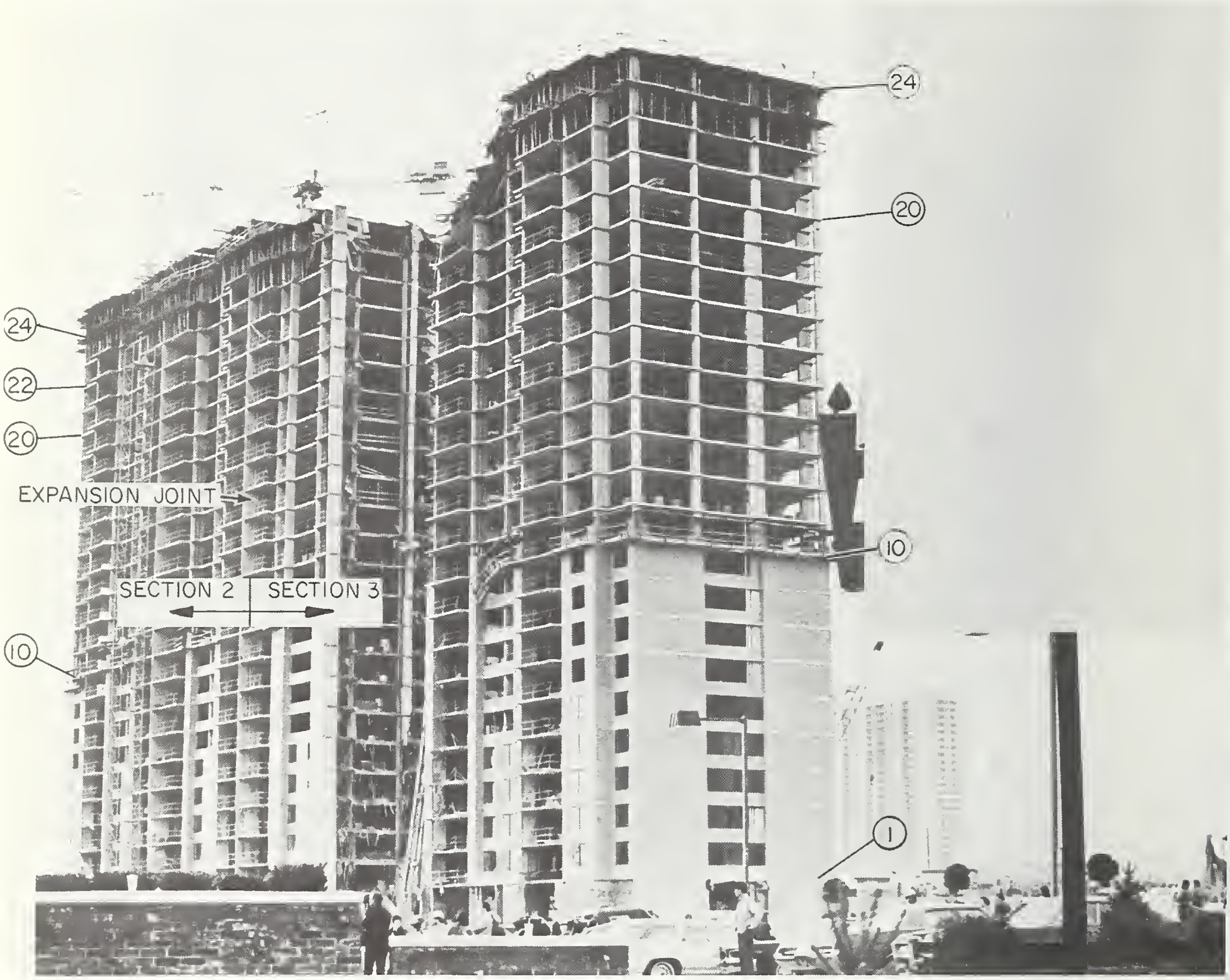
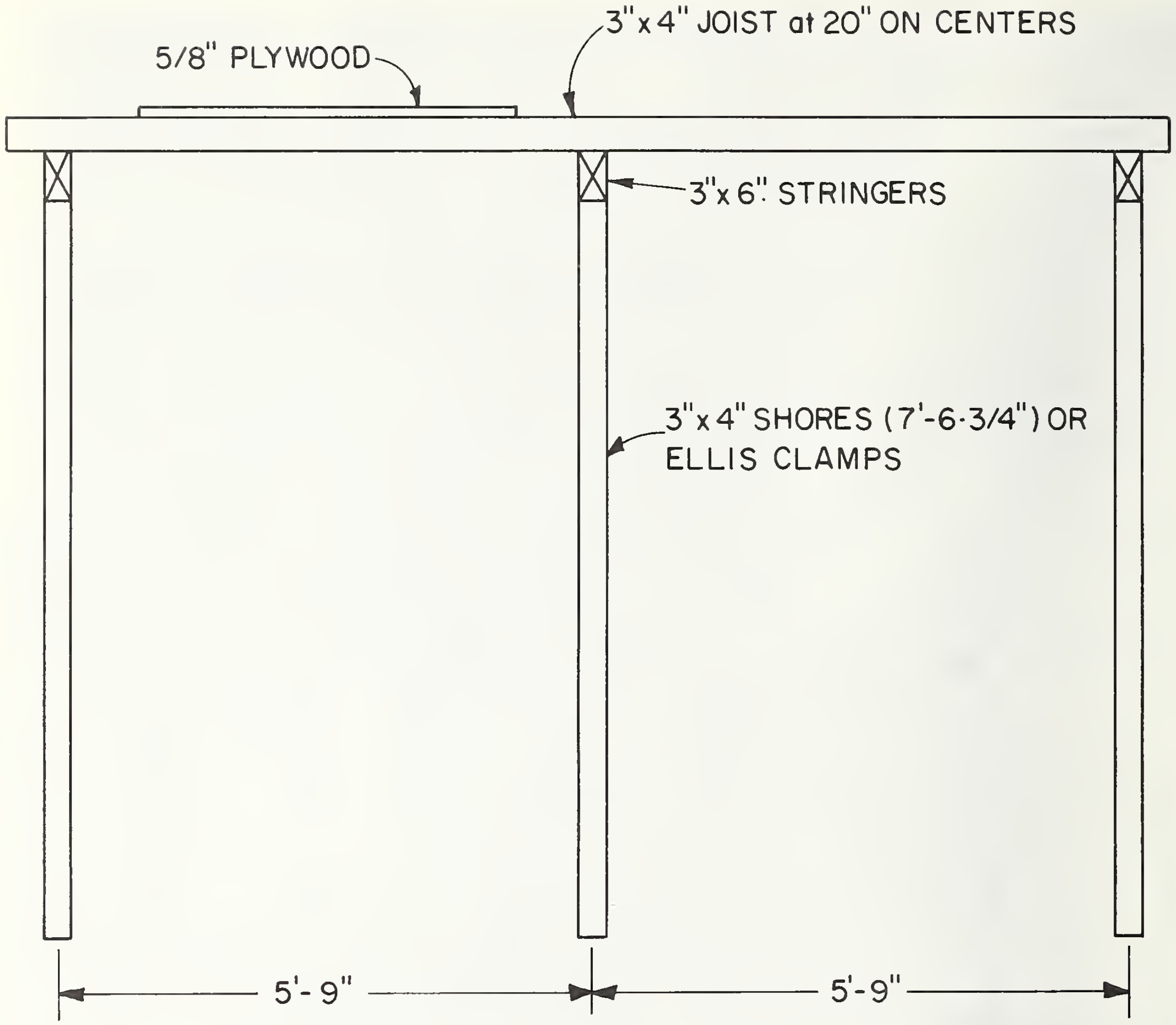


Figure 2.15 General appearance of building A-4 as viewed from the southeast.

Figure 2.16 Formwork in sections 1 and 2 of 24th story, looking east.



Lumber- 1450 f and better
Plywood - Douglas fir, Class I, B/B exterior 2000 f

Figure 2.17 Formwork for a typical floor.

Figure 2.18 Selected details of 24th story forms.

Figure 2.19 Twenty-fourth floor slab in section 3, looking east.

Figure 2.20 Formwork in sections 1 and 2 of 23rd story, looking east.

Figure 2.21 Detail at base of lateral bracing, 23rd floor.

Figure 2.22 Selected details of 23rd story forms.

Figure 2.23 Reshoring in section 2 of 22nd story, looking east.

Figure 2.24 Section 2 of 21st story, looking east.

Figure 2.25 Section 1 and 2 of 20th story, looking east.

Figure 2.26 Section 3 of 20th story, looking east.

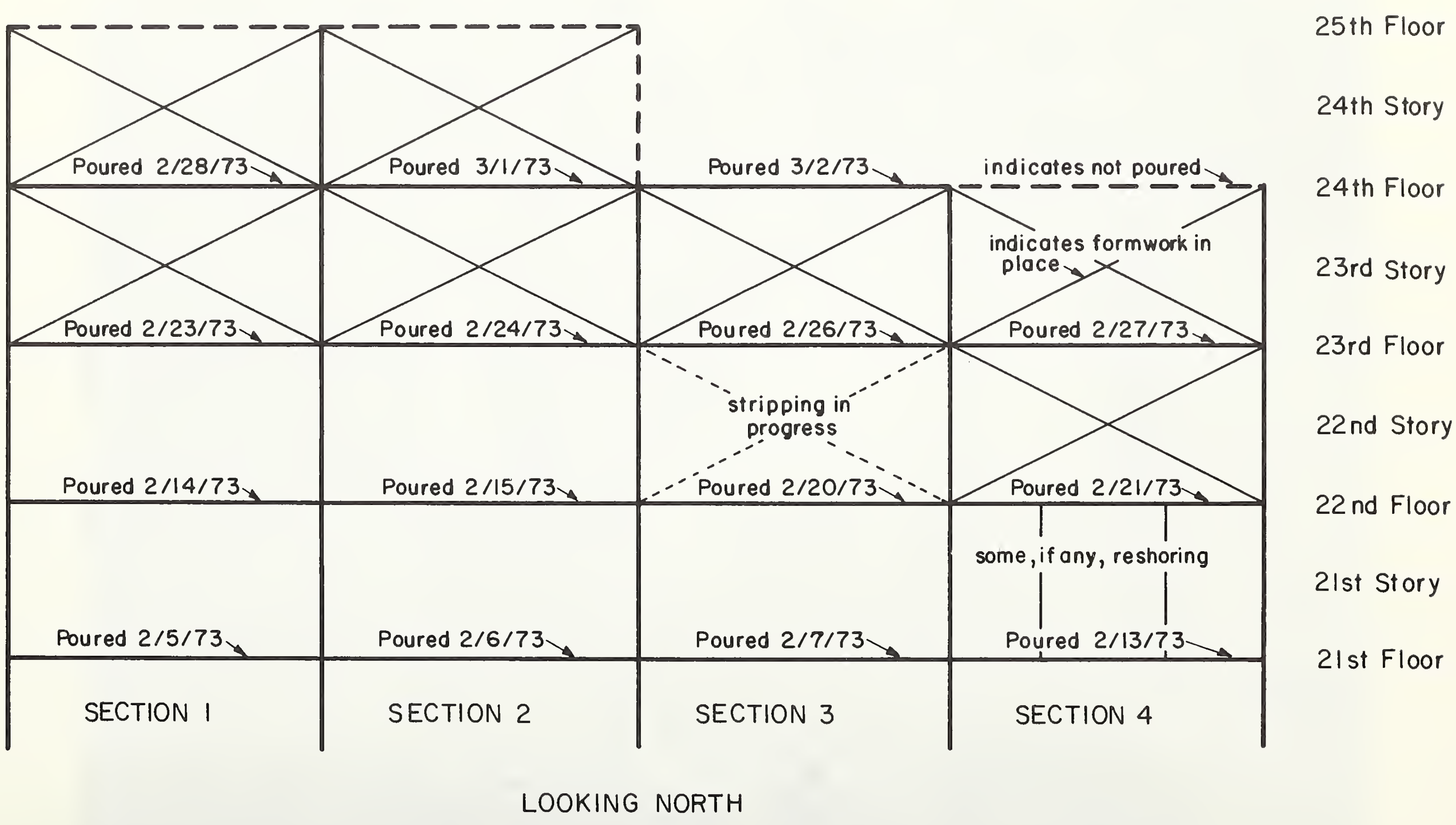


Figure 2.27 Estimated location of forms and reshores at the time of the accident.

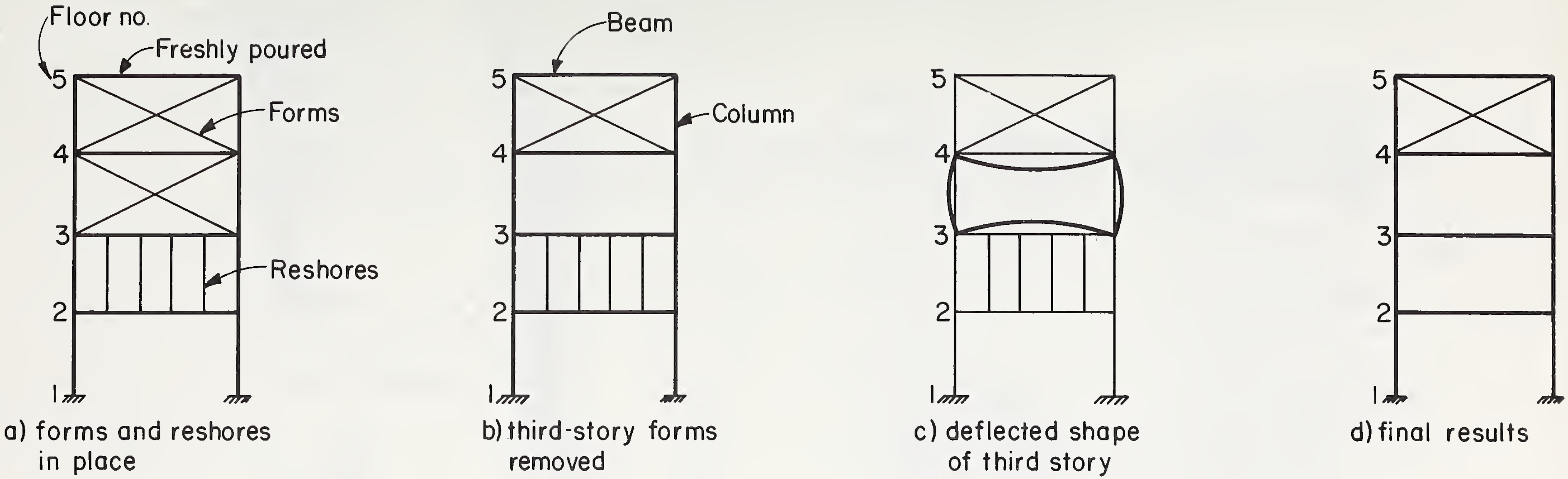


Figure 2.28 Effect of removal of forms.

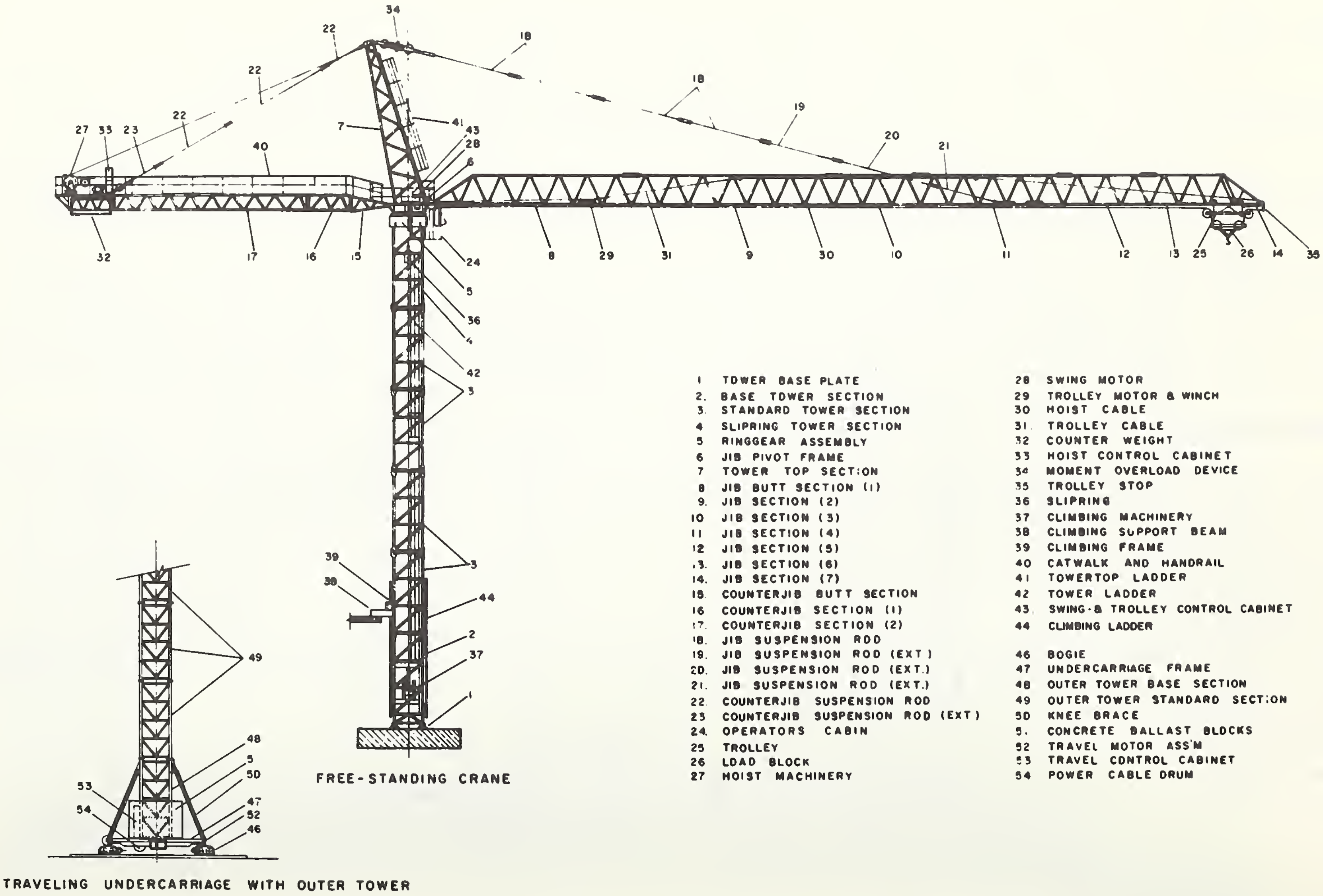


Figure 2.29 Crane terminology.

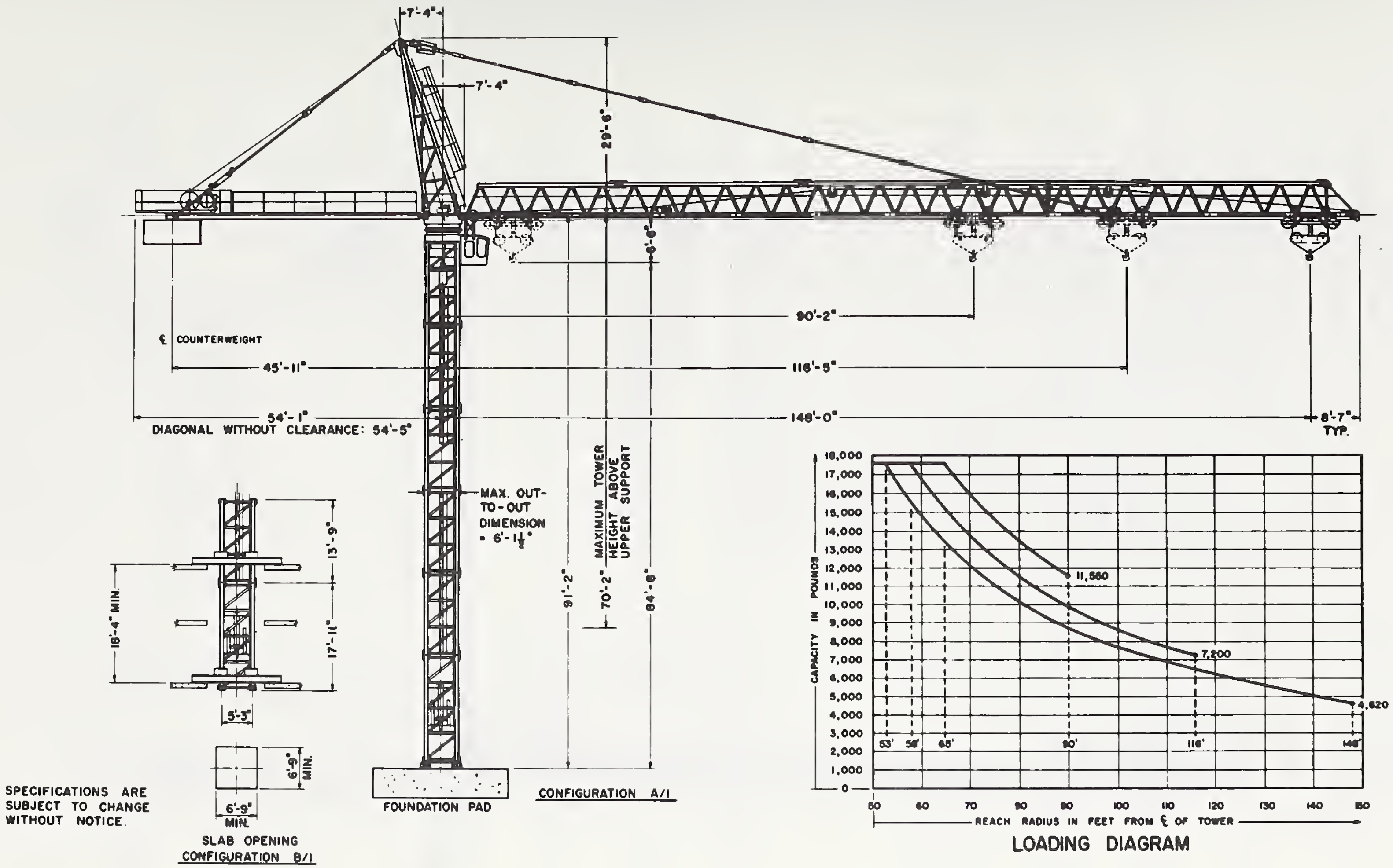


Figure 2.30 Crane no. 1 specifications.

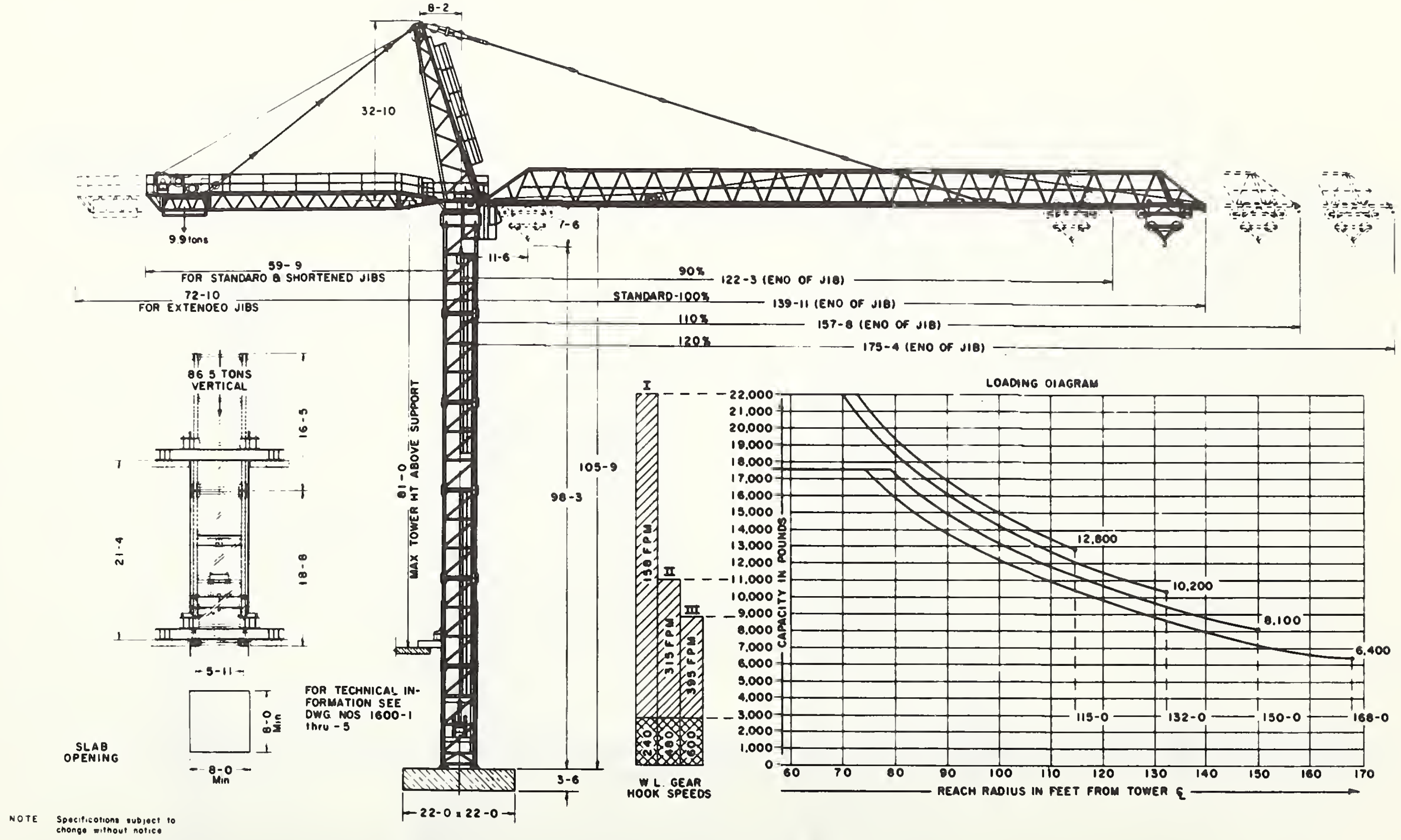


Figure 2.31 Crane no. 2 specifications.

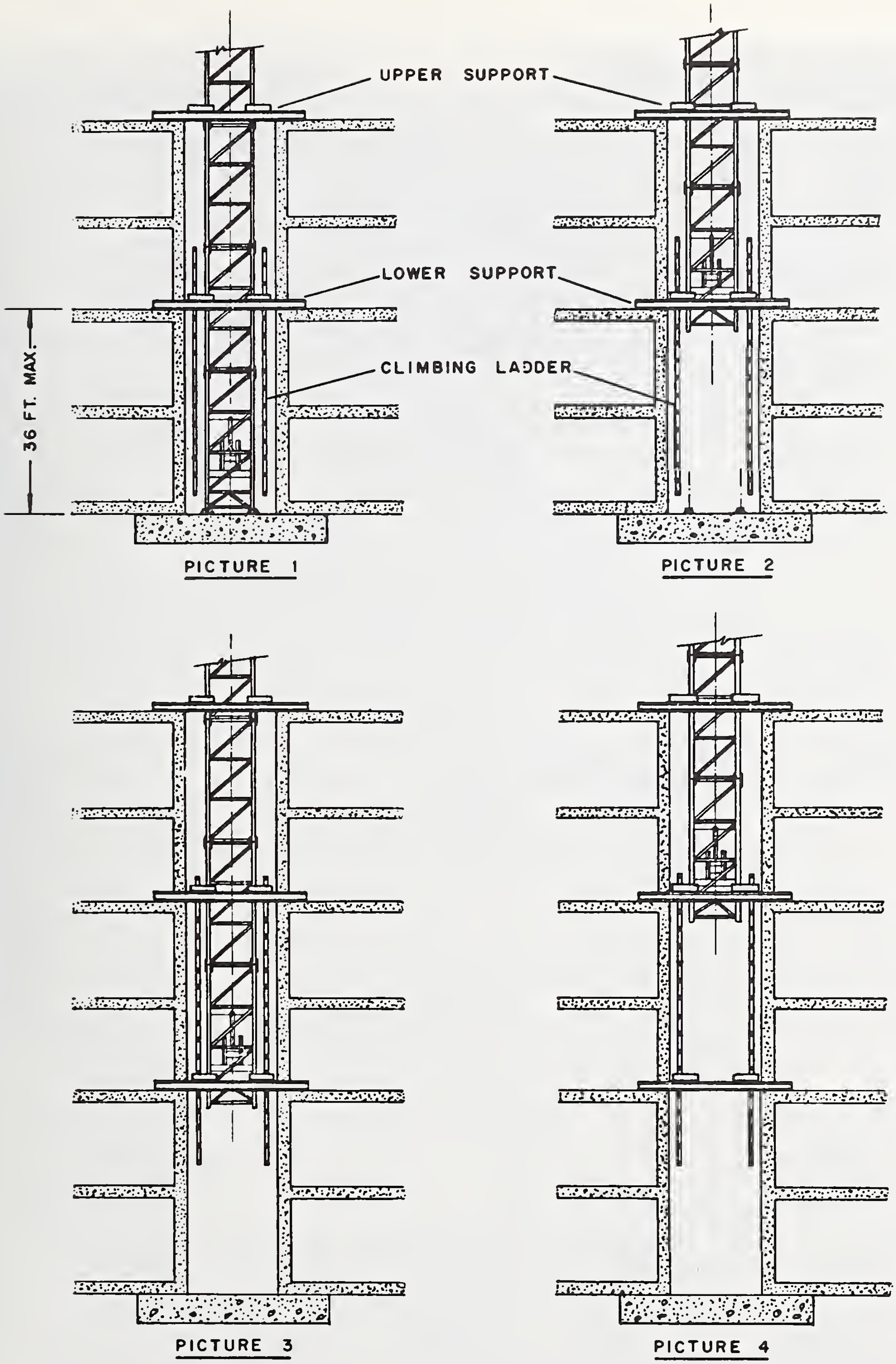


Figure 2.32 Schematic climbing sequence.

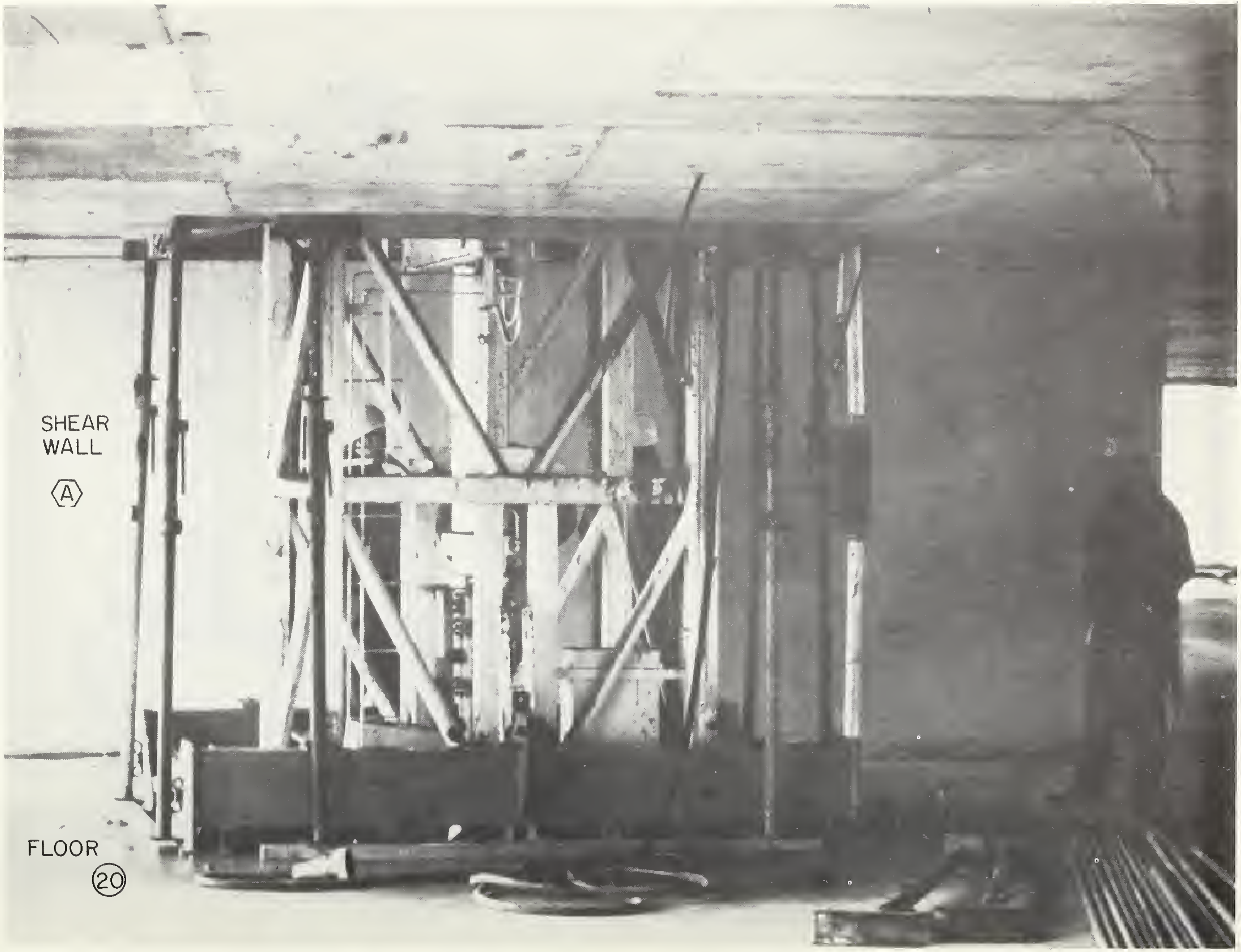


Figure 2.33 Climbing frame for crane no. 1.

Figure 2.34 Upper support for crane no. 1.

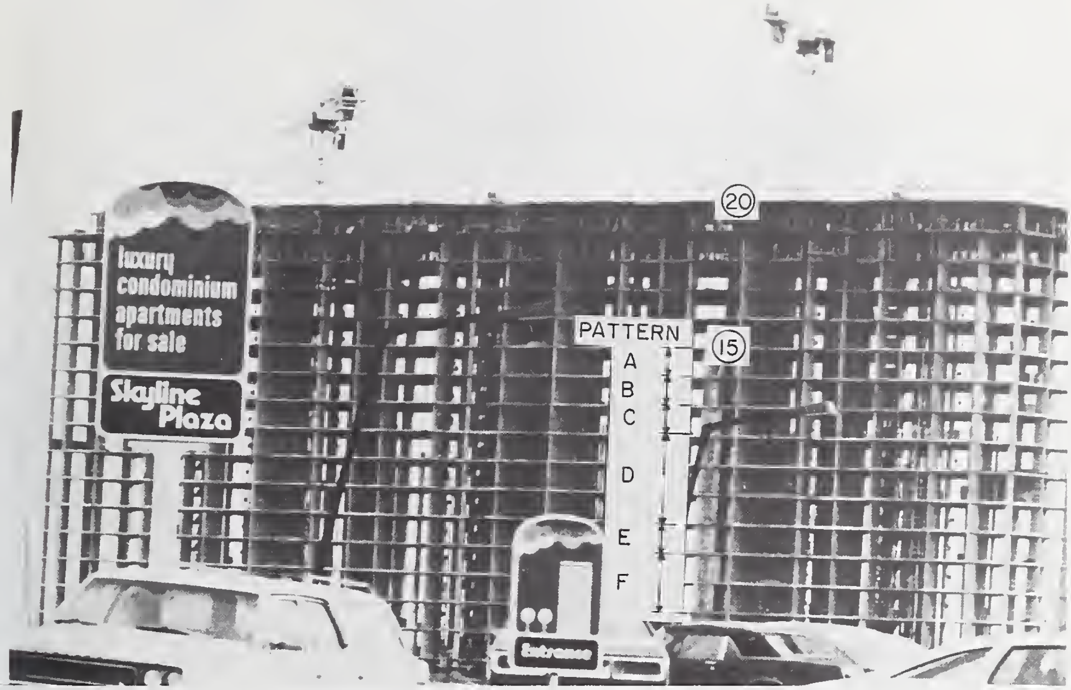


Figure 2.36 Appearance of construction in early February, looking south.

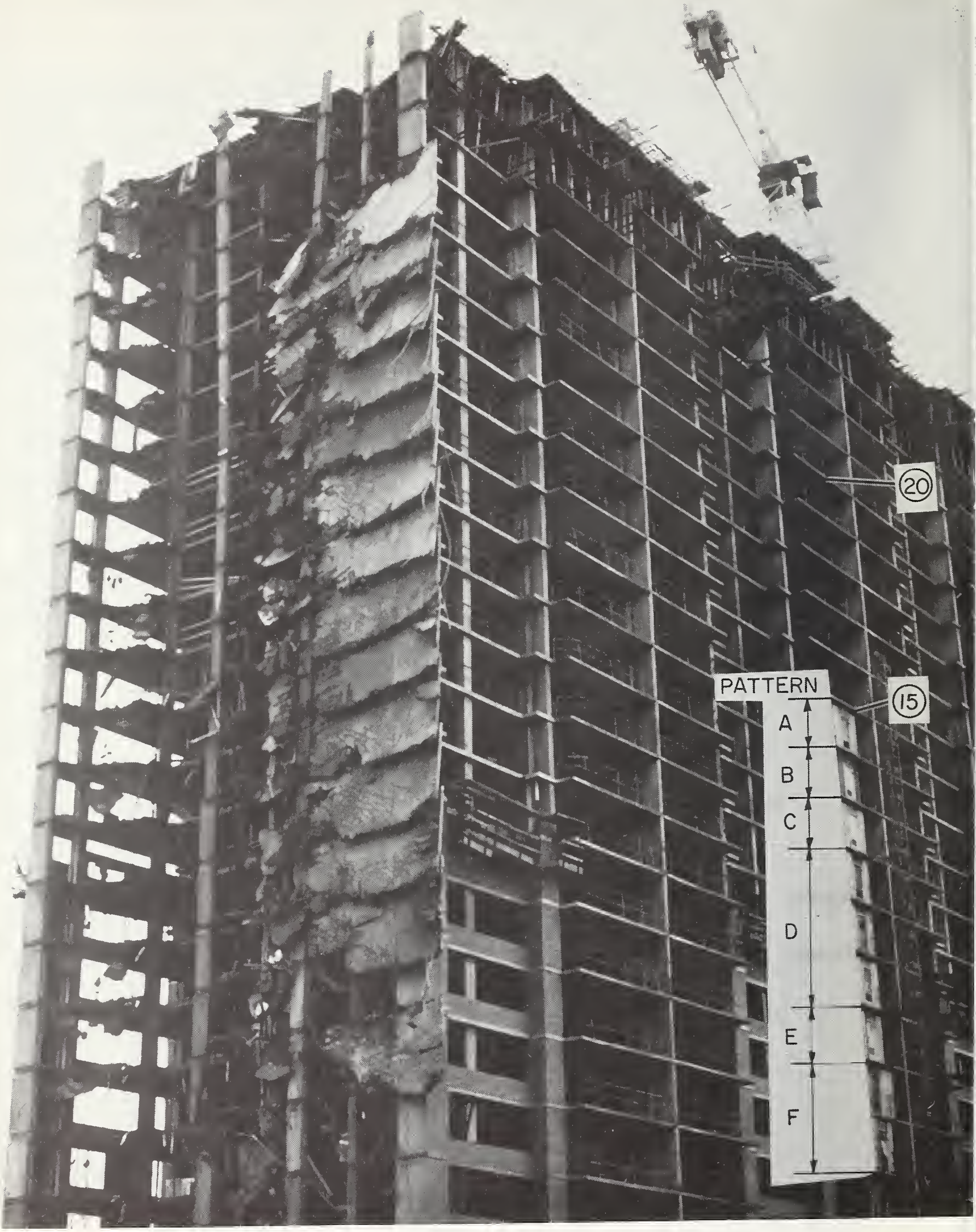


Figure 2.37 Appearance of building A-4 on March 6.

Figure 2.38 Closeup view of figure 2.36 showing crane no. 2 base support.

Figure 2.39 Appearance of construction in February some time after figure 2.36, looking south.

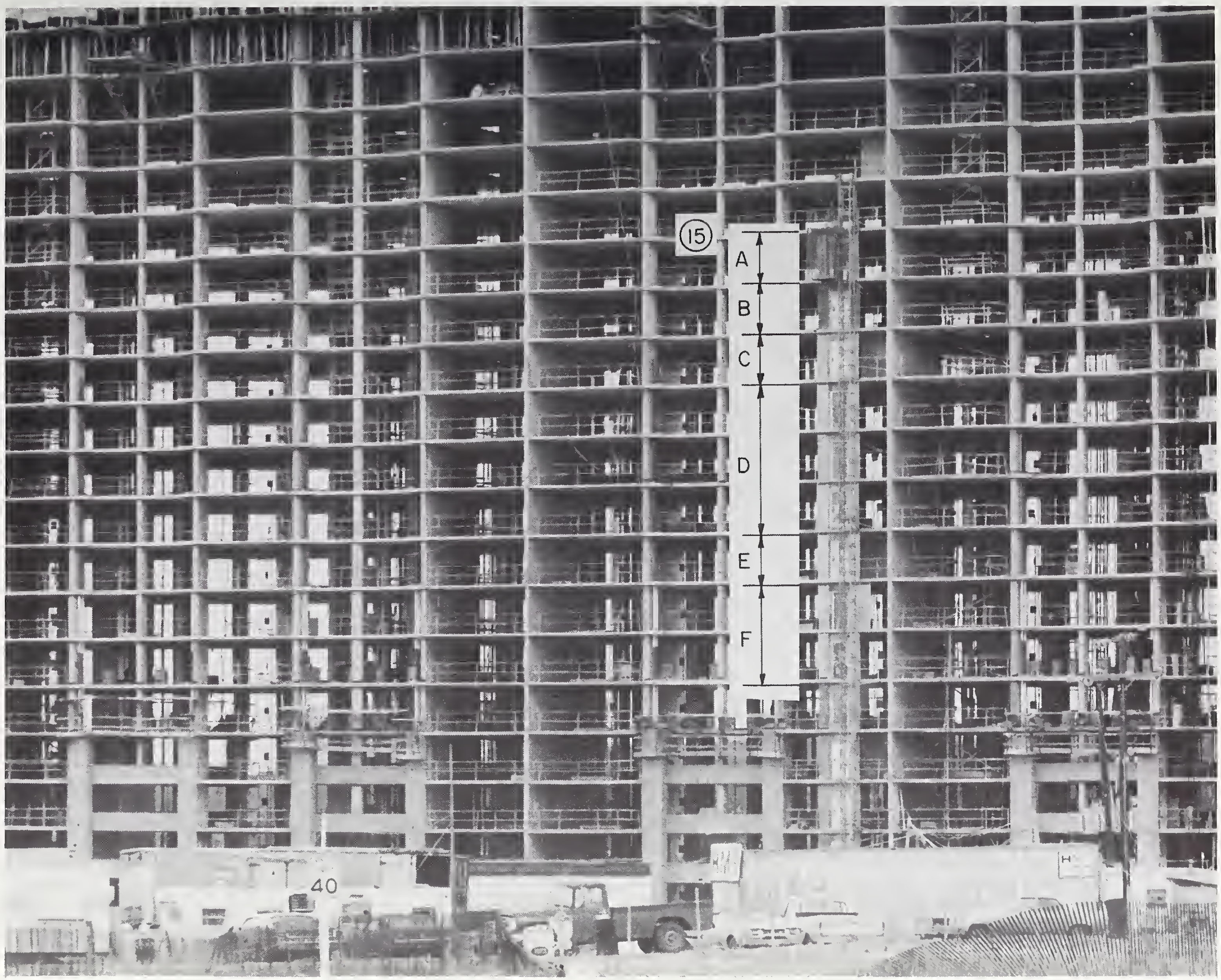


Figure 2.40 Closeup of elevator on north face.

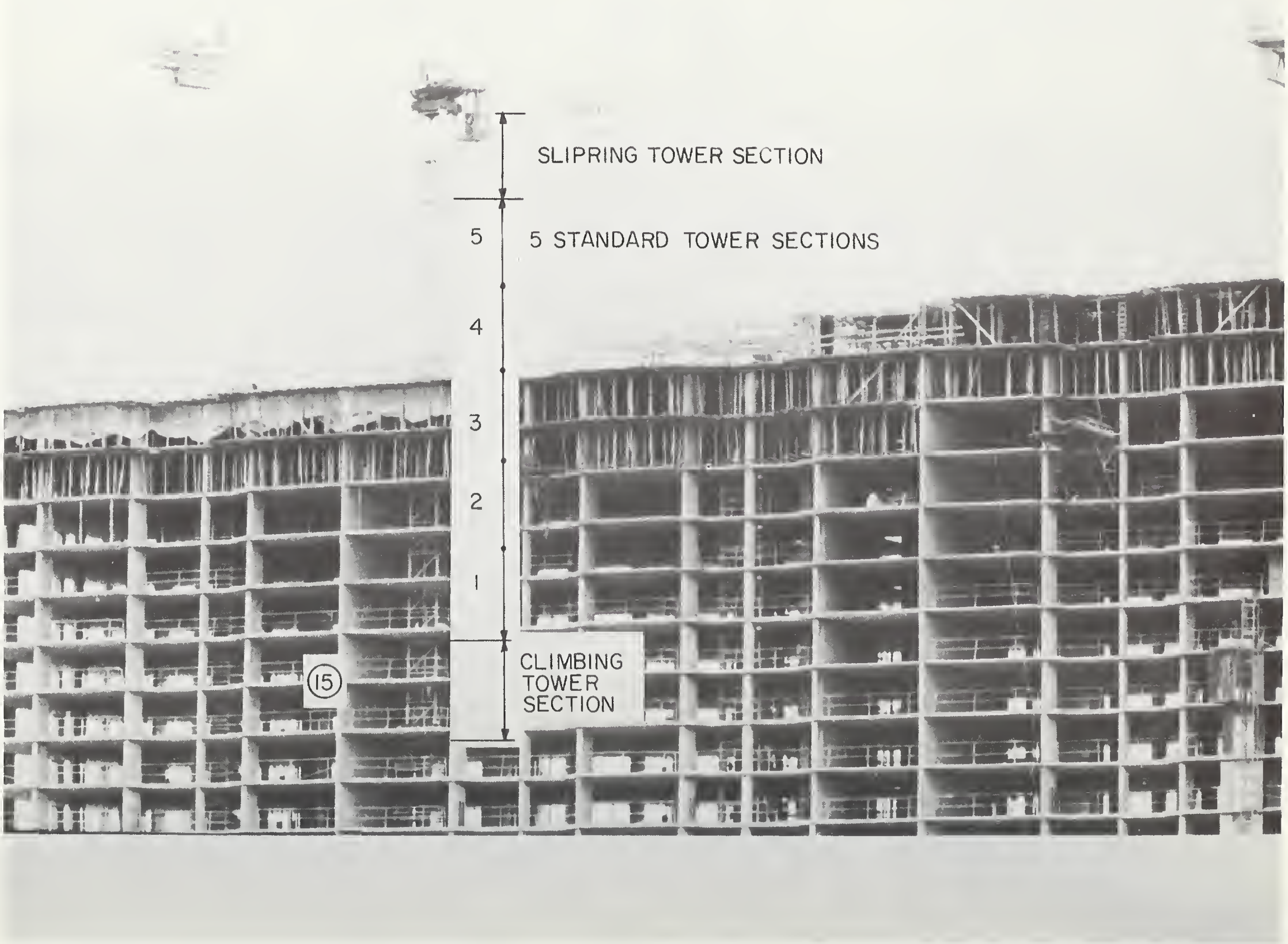


Figure 2.41 Closeup showing crane no. 2 base support.

Figure 2.42 Closeup view showing crane no. 2 base support and shoring under crane.

Figure 2.43 General view of parking garage from building A-4.

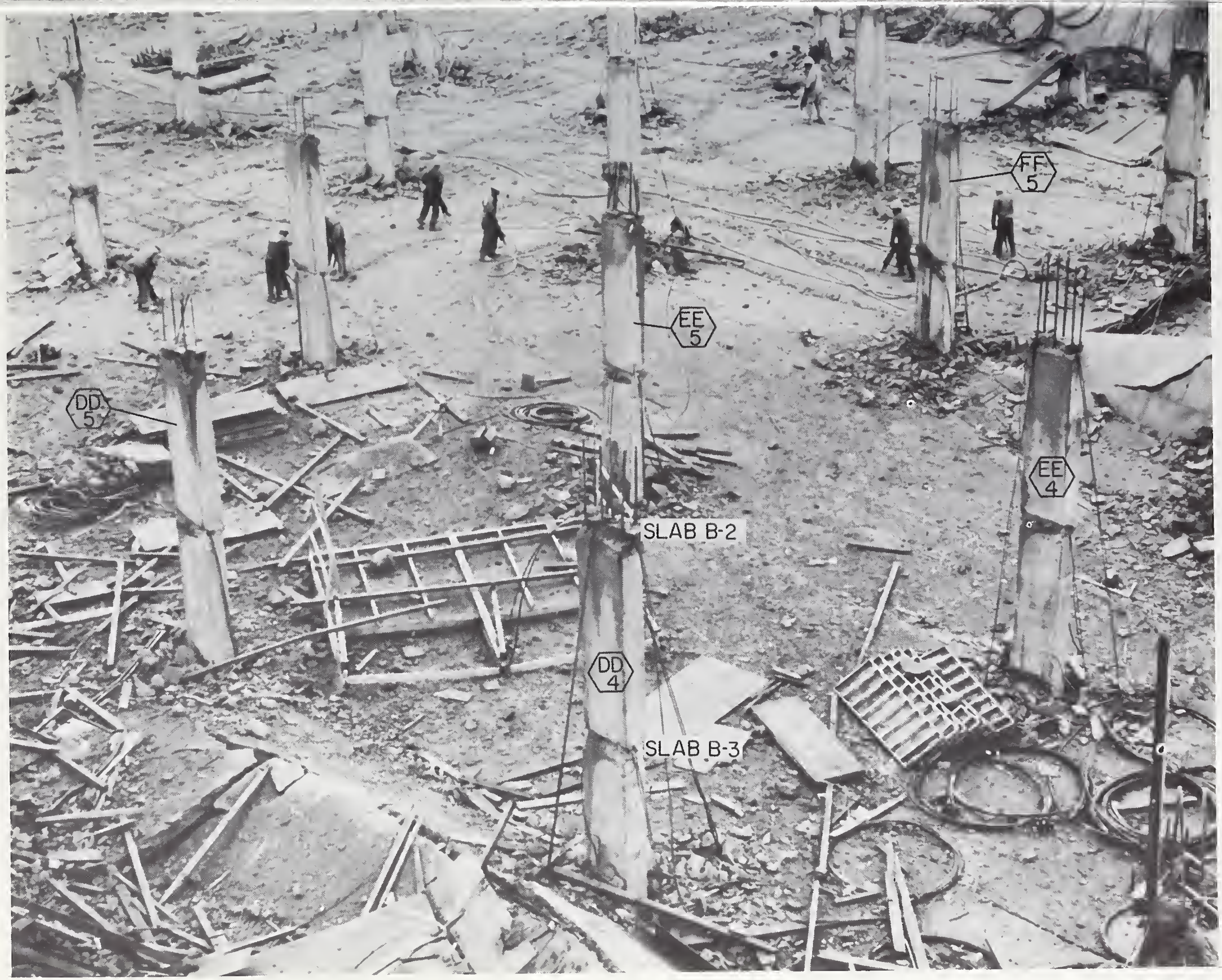


Figure 2.44 Closeup view of columns in figure 2.43.

Figure 2.45 View looking west of slab B-2 between column lines HH and JJ.

Figure 2.46 Edge of slab B-2 at columns JJ-5 and JJ-6.

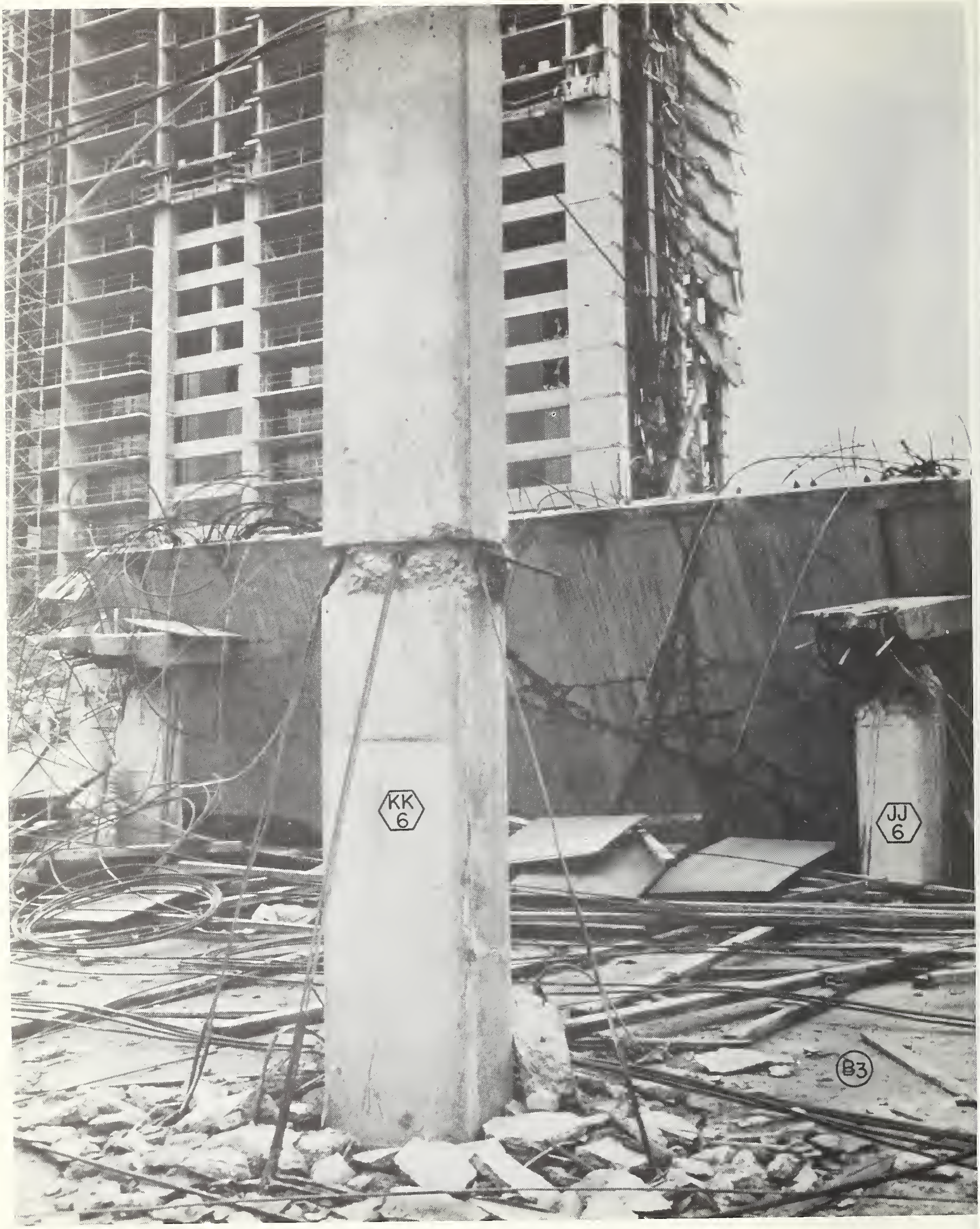


Figure 2.47 Typical column in the parking garage.

Figure 2.48 Typical view of column base.

Figure 2.49 View of garage area looking northwest showing crane no. 2 and column corbels on building A-4.

Figure 2.50 View looking north of ramp area and column corbels on building A-4.

Figure 2.51 View looking west of area between the ramp and building A-4.

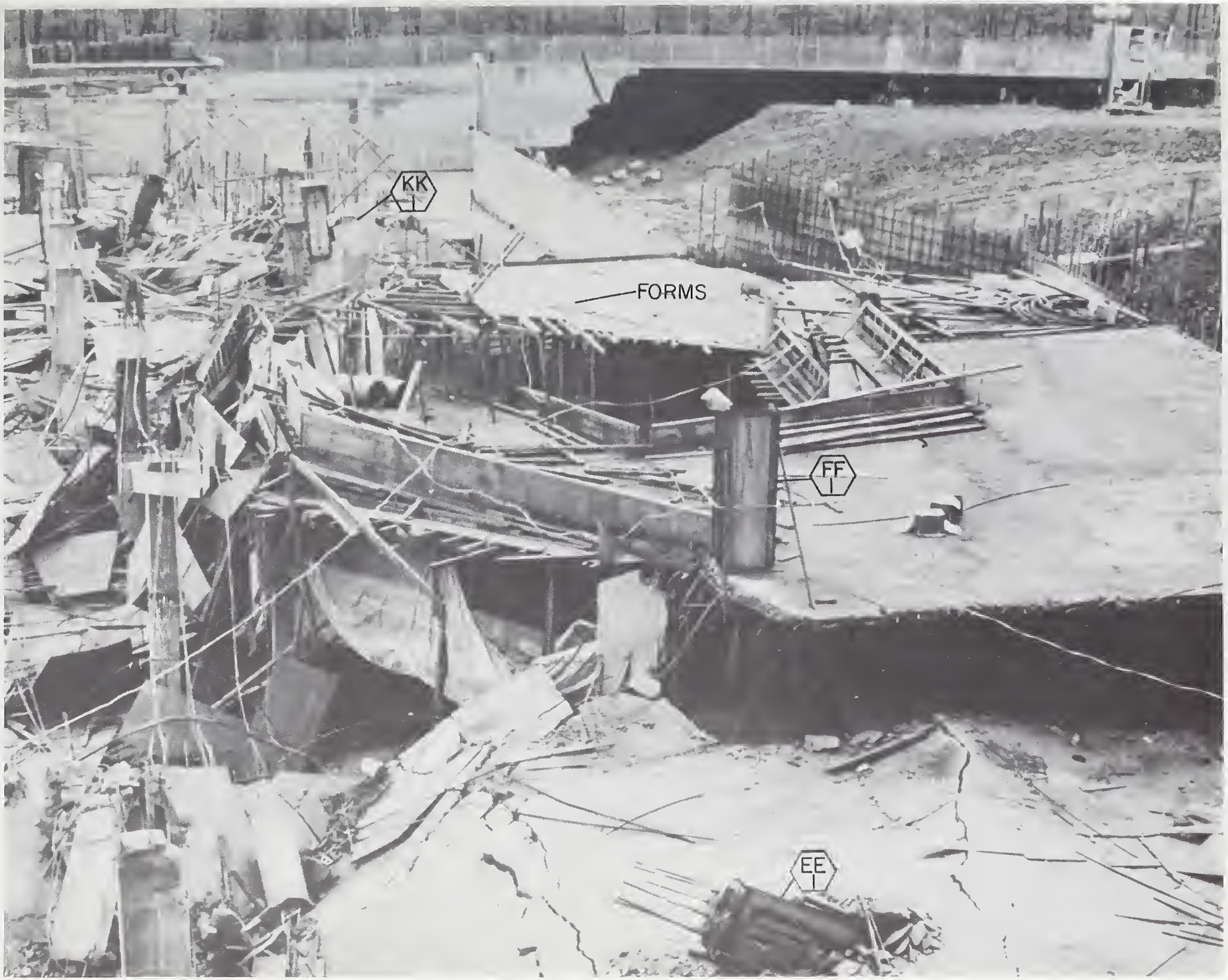


Figure 2.52 View of ramp area looking south.

Figure 2.53 View looking west showing southwest area of parking garage.

Figure 2.54 Slab support on western wall of parking garage.

Figure 2.55 Position of fallen crane near column DD-2 looking south.

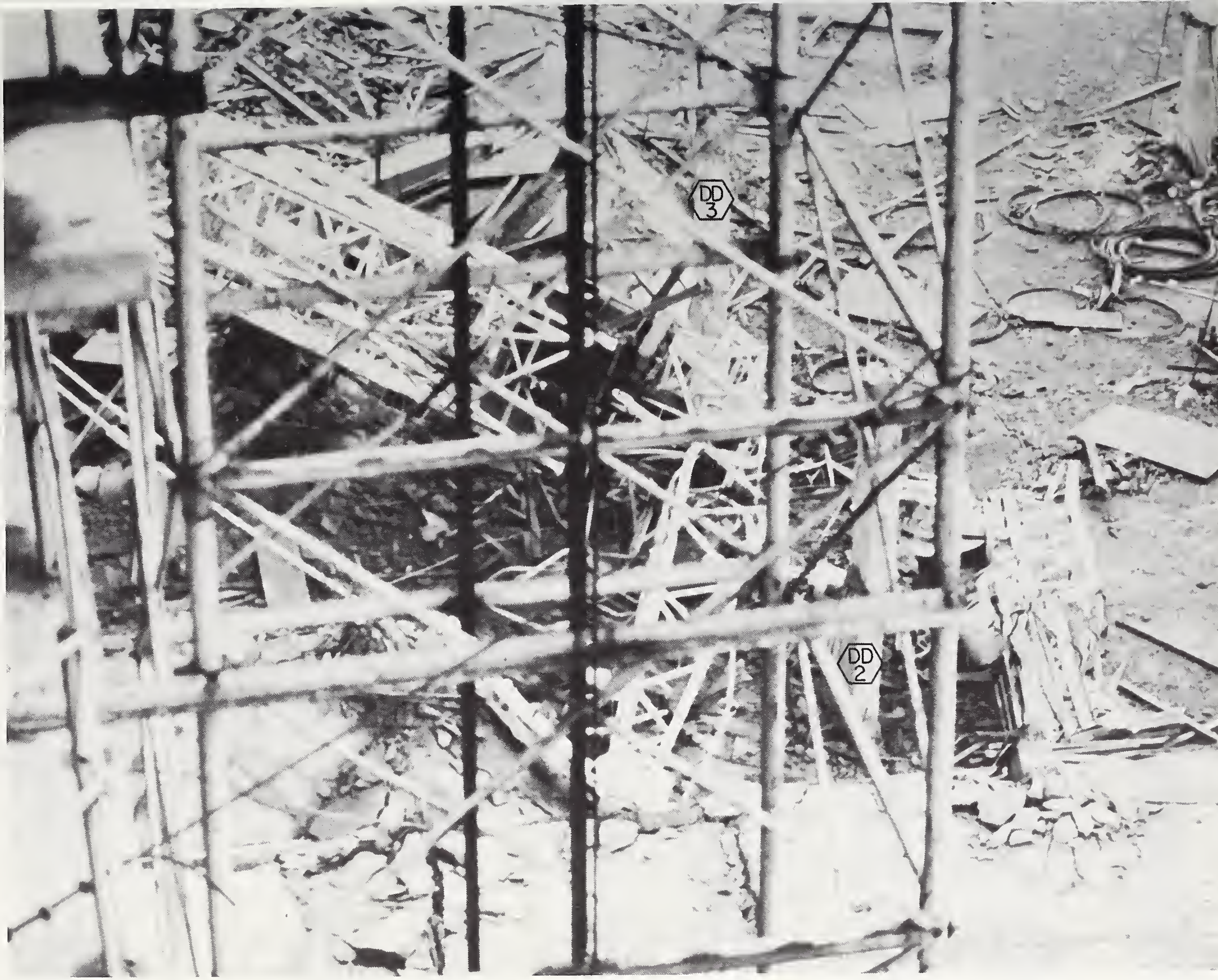


Figure 2.56 Position of fallen crane near column DD-2 looking east.

	Section 1	Section 2	Section 3	Section 4
Columns Cast	3/ 2/73			
7 day cylinder strength	-			
28 day cylinder strength	-			
Slab Cast	2/28/73	3/ 1/73	3/ 2/73	
7 day cylinder strength	2095 psi	2825 psi	-	
28 day cylinder strength	-	-	-	
24th Floor				
Columns Cast	2/26/73	2/27/73	2/28/73	3/ 1/73
7 day cylinder strength	2775 psi	2085 psi	2695 psi	2325 psi
28 day cylinder strength	-	-	3610 psi	3235 psi
Slab Cast	2/23/73	2/24/73	2/26/73	2/27/73
7 day cylinder strength	1380 psi	-	2290 psi	2575 psi
28 day cylinder strength	-	-	3405 psi	3750 psi
23rd Floor				
Columns Cast	2/20/73	2/21/73	2/23/73	2/23/73
7 day cylinder strength	-	2740 psi	2460 psi	2460 psi
28 day cylinder strength	-	-	3125 psi	3125 psi
Slab Cast	2/14/73	2/15/73	2/20/73	2/21/73
7 day cylinder strength	2220 psi	1540 psi	2280 psi	2115 psi
28 day cylinder strength	-	-	3210 psi	3205 psi
22nd Floor				
Columns Cast	2/ 7/73	2/13/73	2/14/73	2/15/73
7 day cylinder strength	3185 psi	2610 psi	2325 psi	2210 psi
28 day cylinder strength	4405 psi	-	3385 psi	3565 psi
Slab Cast	2/ 5/73	2/ 6/73	2/ 7/73	2/13/73
7 day cylinder strength	2690 psi	3160 psi	2110 psi	2070 psi
28 day cylinder strength	3195 psi	3520 psi	3025 psi	3345 psi
21st Floor				
Slab Cast			1/10/73	1/11/73
7 day cylinder strength			3105 psi	2185 psi
28 day cylinder strength			4270 psi	3130 psi
17th Floor				

Figure 3.1 Casting dates and laboratory cylinder strengths

Floor No.	Number of Cores Tested	Age at Time of Collapse, Days	Age at Time of Test, Days	Average Strength at Time of Test, psi	Range of Strength psi	Average Density, lb/cu ft
4" Diameter Core Compression Tests						
23	12	4	16	2295	1690-3360	119
22	13	10	25	1960	1390-2840	118
21	12	23	38	2470	1810-3150	117
17	12	51	66	2780	2450-3420	124
4" Diameter Core Split Cylinder Tests						
23	3	4	23	295	255-320	-
22	3	10	29	320	305-335	-

Figure 3.2 Results of strength tests on 4-in diameter cores.

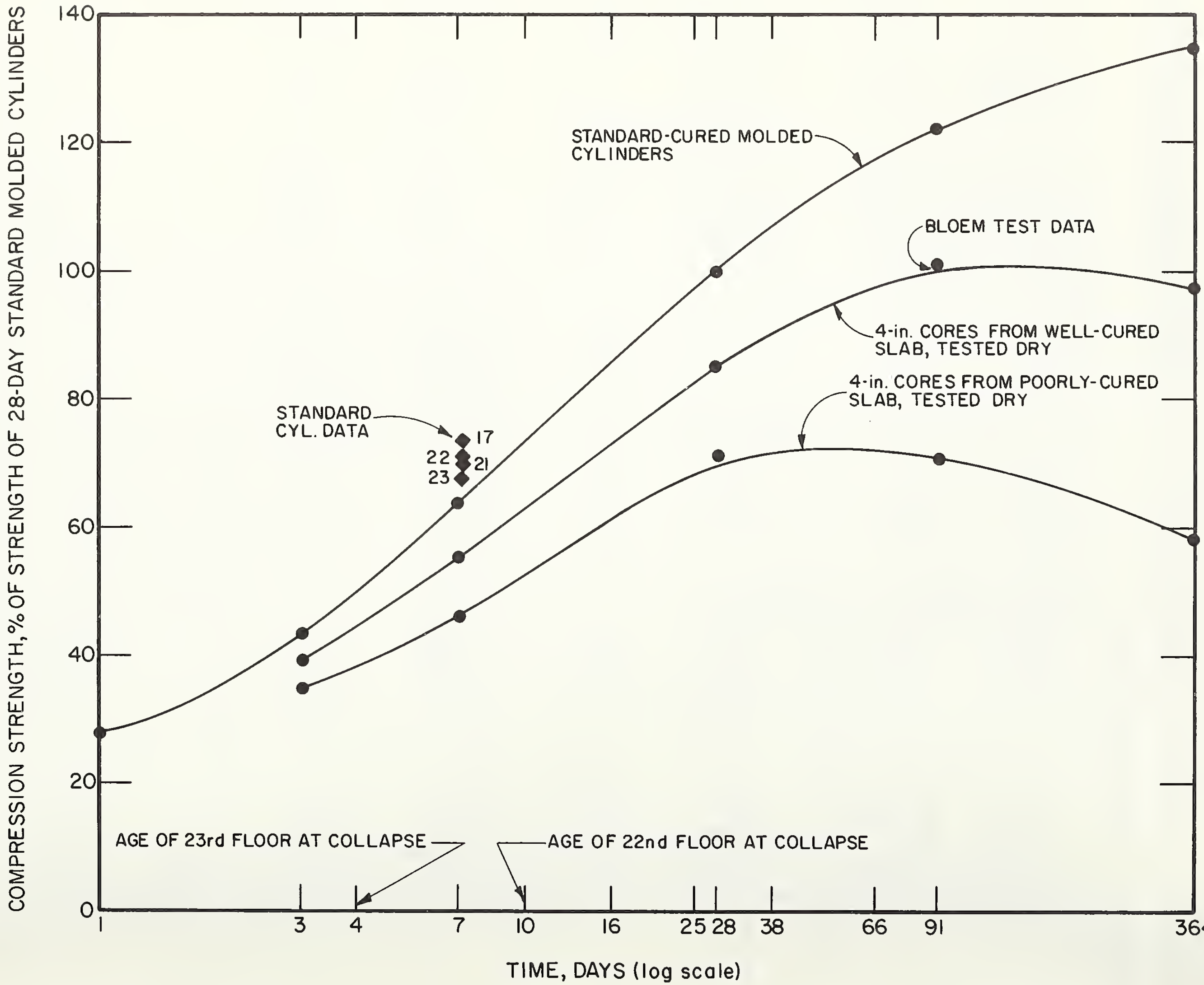


Figure 3.3 Strength measures for concrete cores and cylinders.

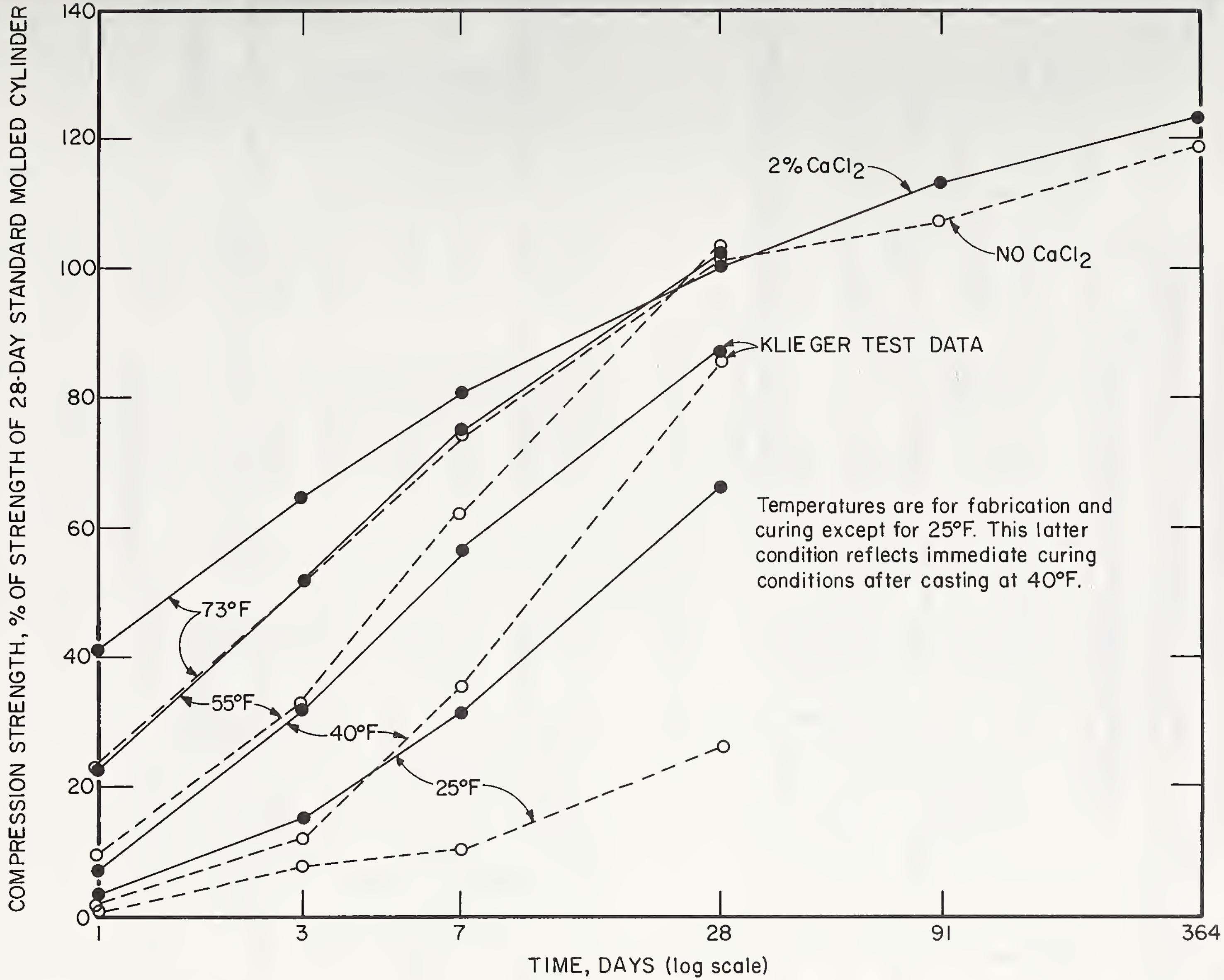


Figure 3.4 Development of concrete strength under various curing temperatures.

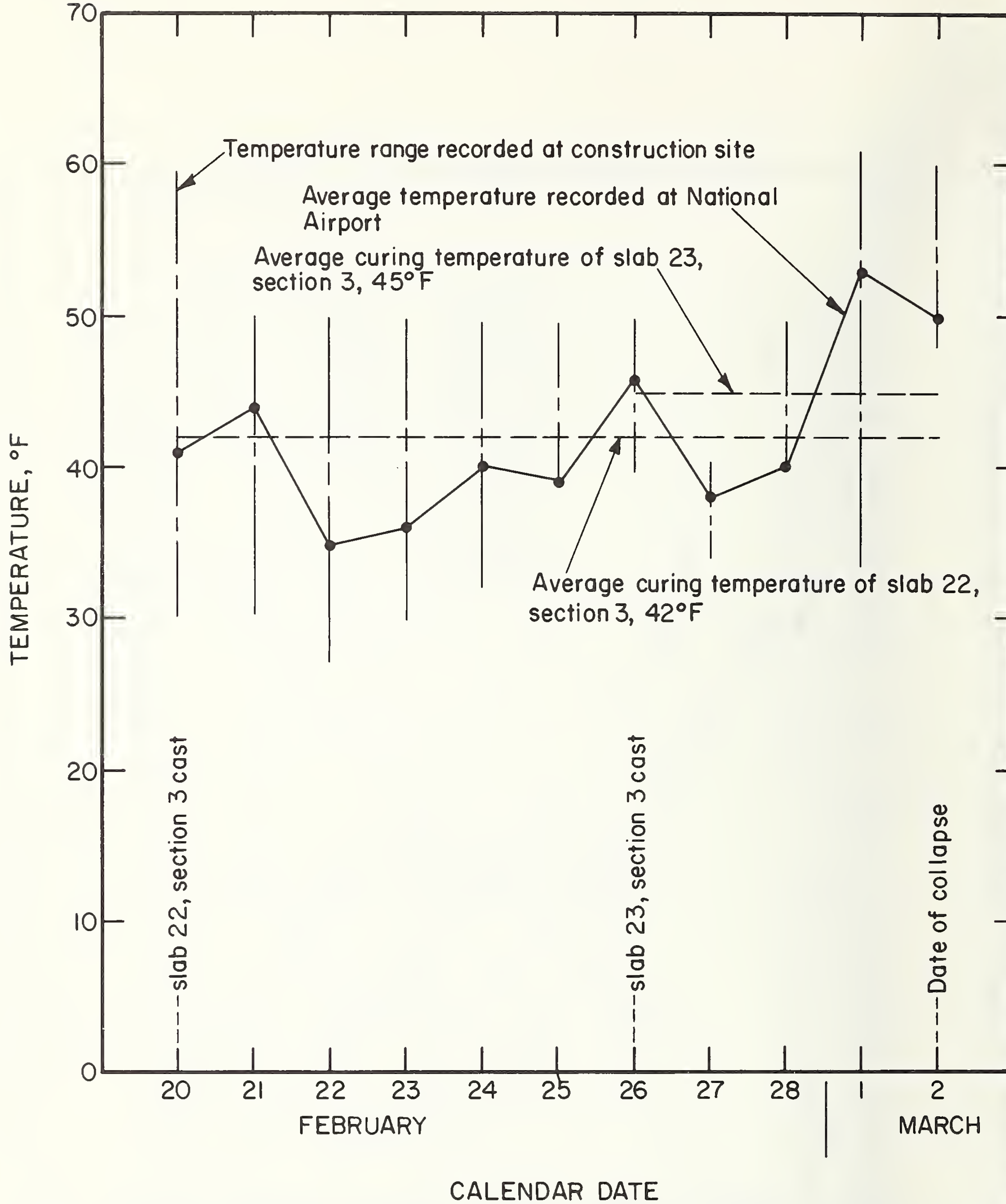


Figure 3.5 Temperature history for section 3 or 22nd and 23rd floor slabs.

Reinforcing Bars Removed from Floor Slabs
from Collapsed Section of Building

Sample No.	Size	Yield Load, lbs.	Maximum Load, lbs.	Yield Strength, psi	Tensile Strength, psi	% Elongation
1	#4	13,500	22,800	68,700	116,000	8.2
2	#4	12,600	22,100	64,100	112,500	8.9
3	#4	14,000	21,600	71,300	110,000	14.1
4	#5	22,000	34,000	71,700	110,800	9.4
5	#5	21,000	33,800	68,400	110,200	9.4
6	#7	38,400	62,000	63,800	103,100	8.6

New Reinforcing Bars Obtained from
The Jobsite

Sample No.	Size	Yield Load, lbs.	Maximum Load,-lbs.	Yield Strength, psi	Tensile Strength, psi	% Elongation
7	#4	13,500	21,750	68,700	110,700	13.3
8	#4	13,000	21,300	66,200	108,500	13.3
9	#5	21,400	34,800	69,700	113,400	13.9
10	#5	21,600	35,600	70,400	116,000	13.3
11	#7	39,500	64,200	65,700	109,300	11.2
12	#7	39,400	65,000	65,500	108,900	10.9

Figure 3.6 Results of reinforcing bar tests.

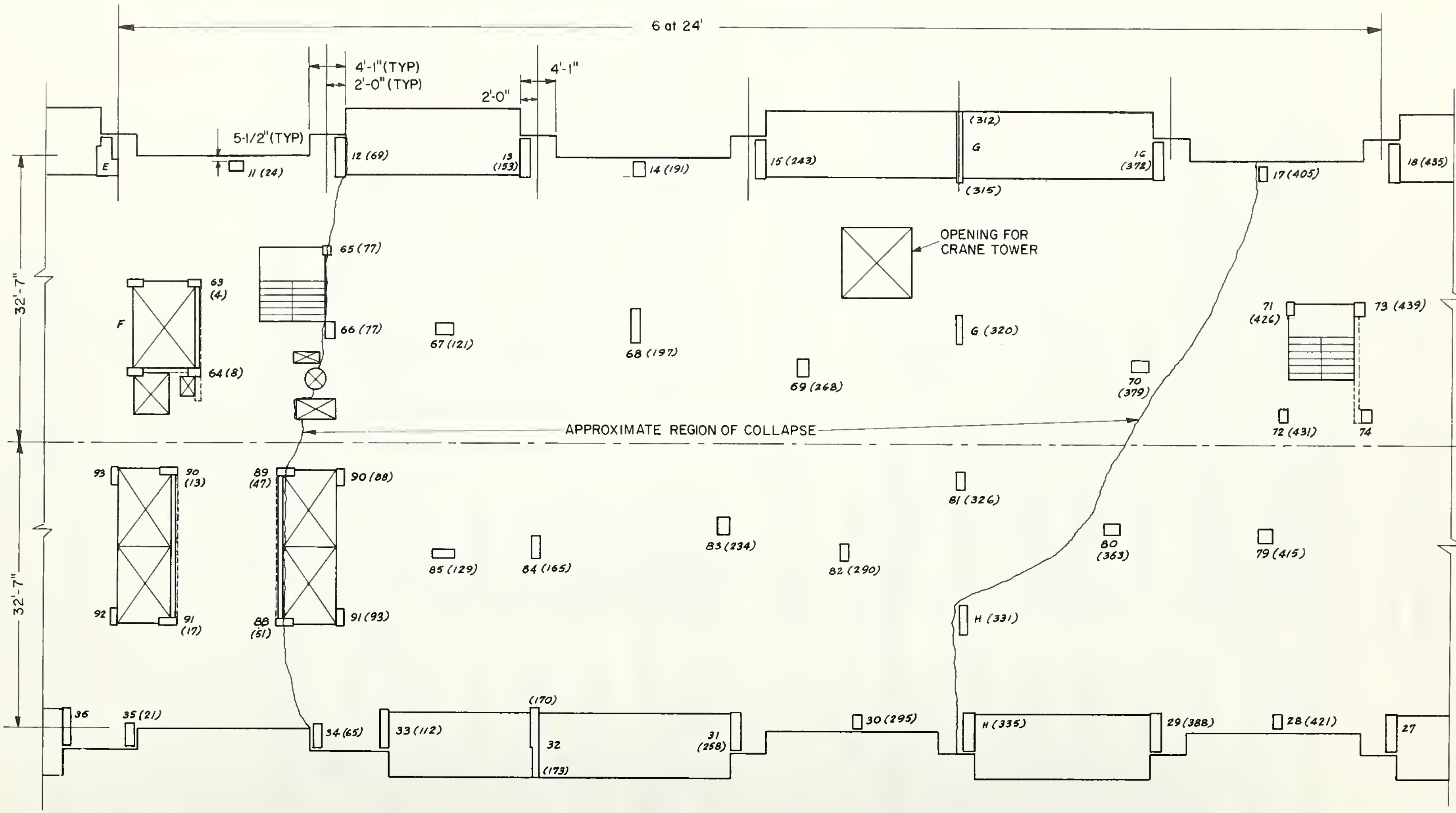


Figure 4.1 PLAN OF FLOOR SLAB AT 22nd AND 23rd FLOORS SHOWING APPROXIMATE AREA OF COLLAPSE

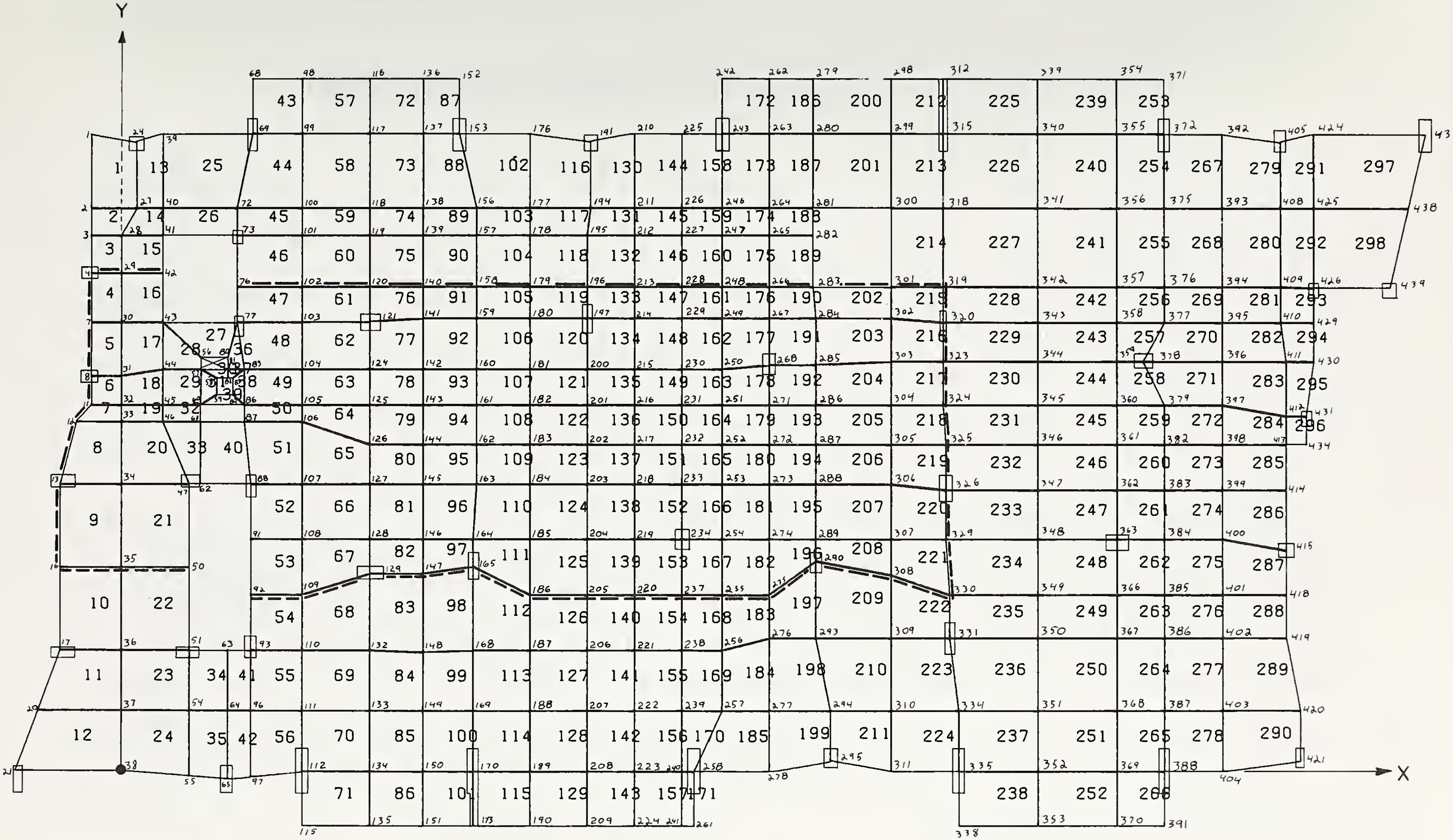


Figure 4.2 FINITE ELEMENT GRID FOR SLAB ANALYSIS

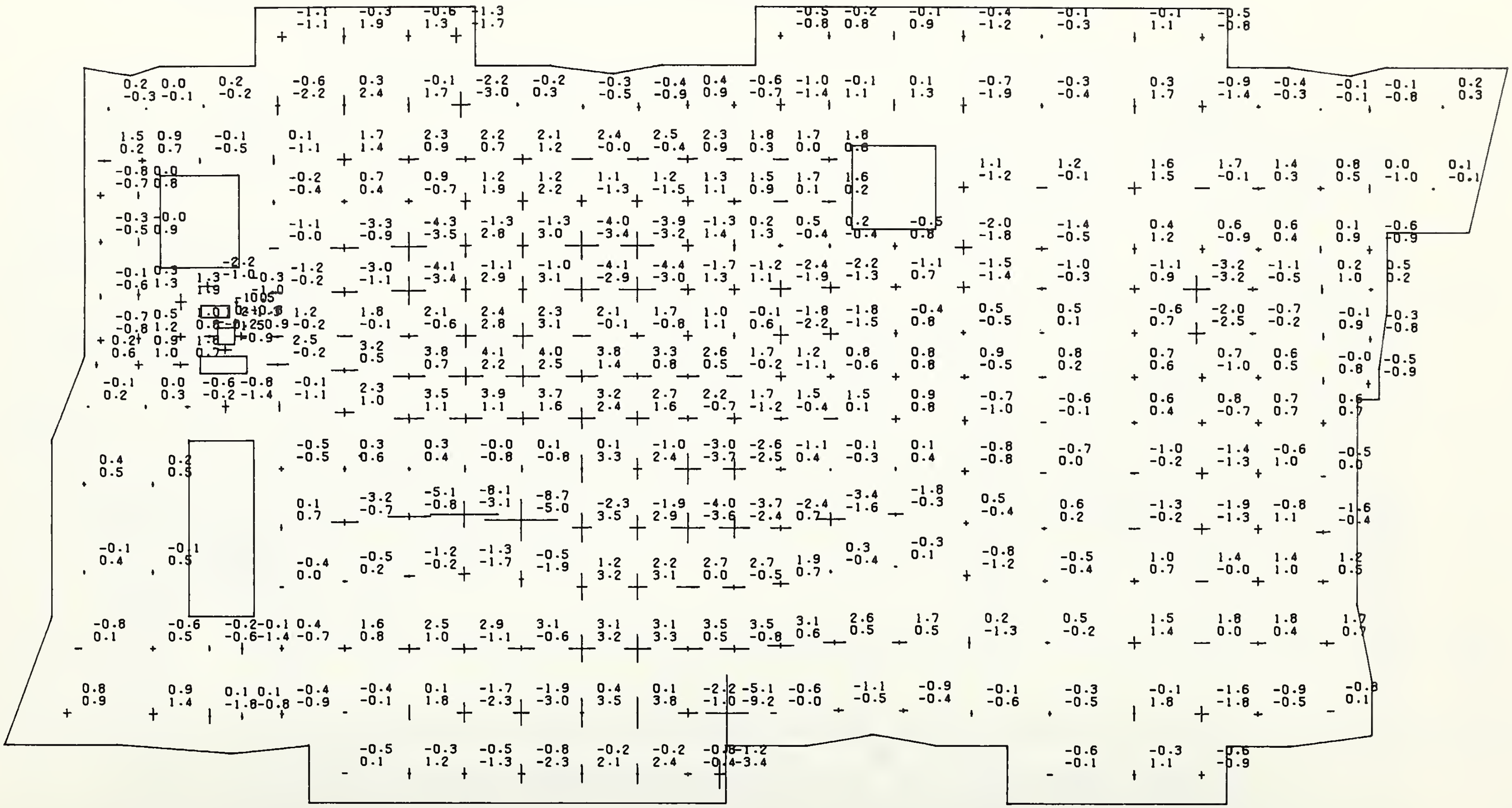


Figure 4.3 INTERNAL MOMENTS IN SLAB-CASE I

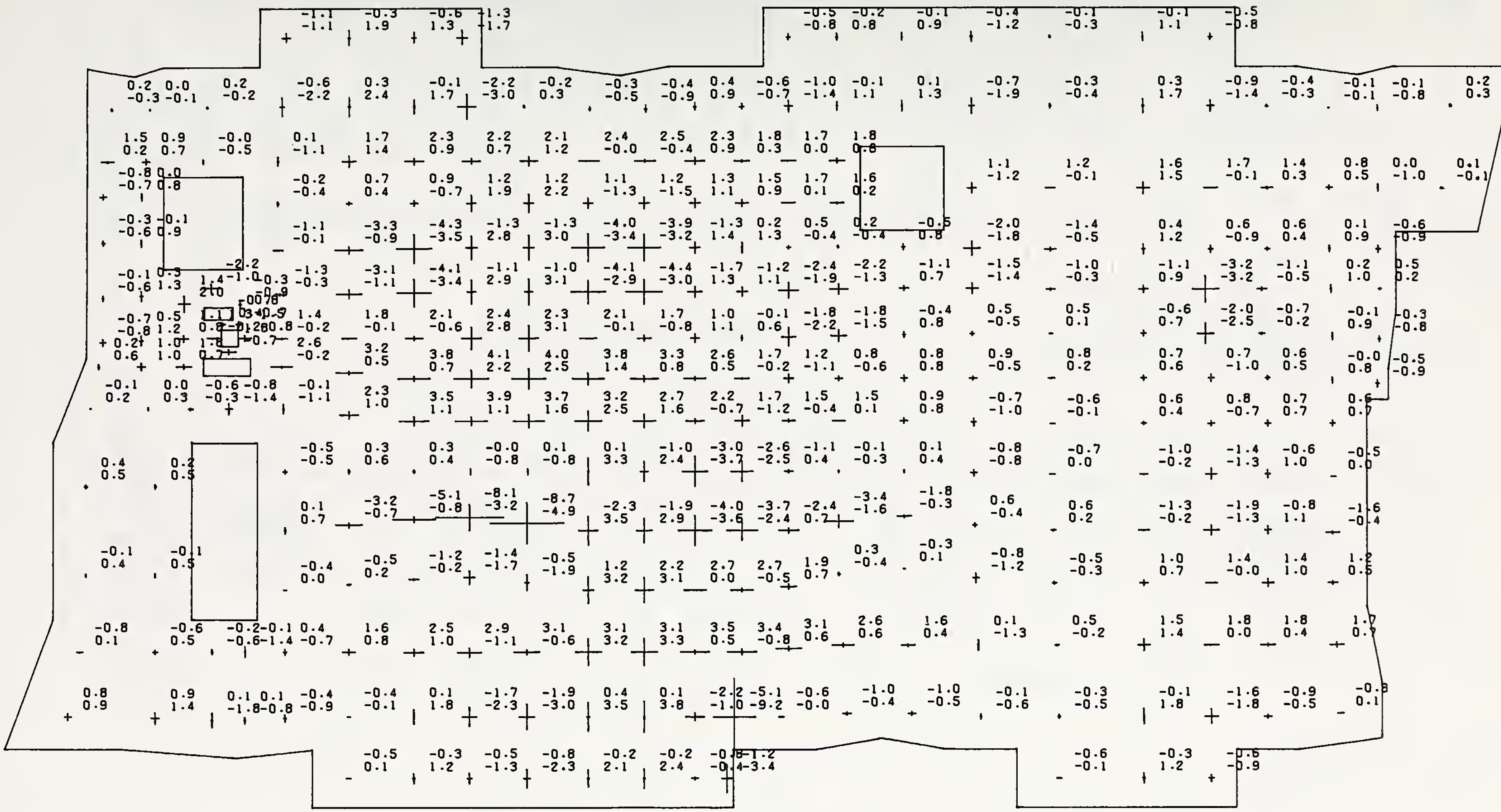


Figure 4.4 INTERNAL MOMENTS IN SLAB-CASE 2

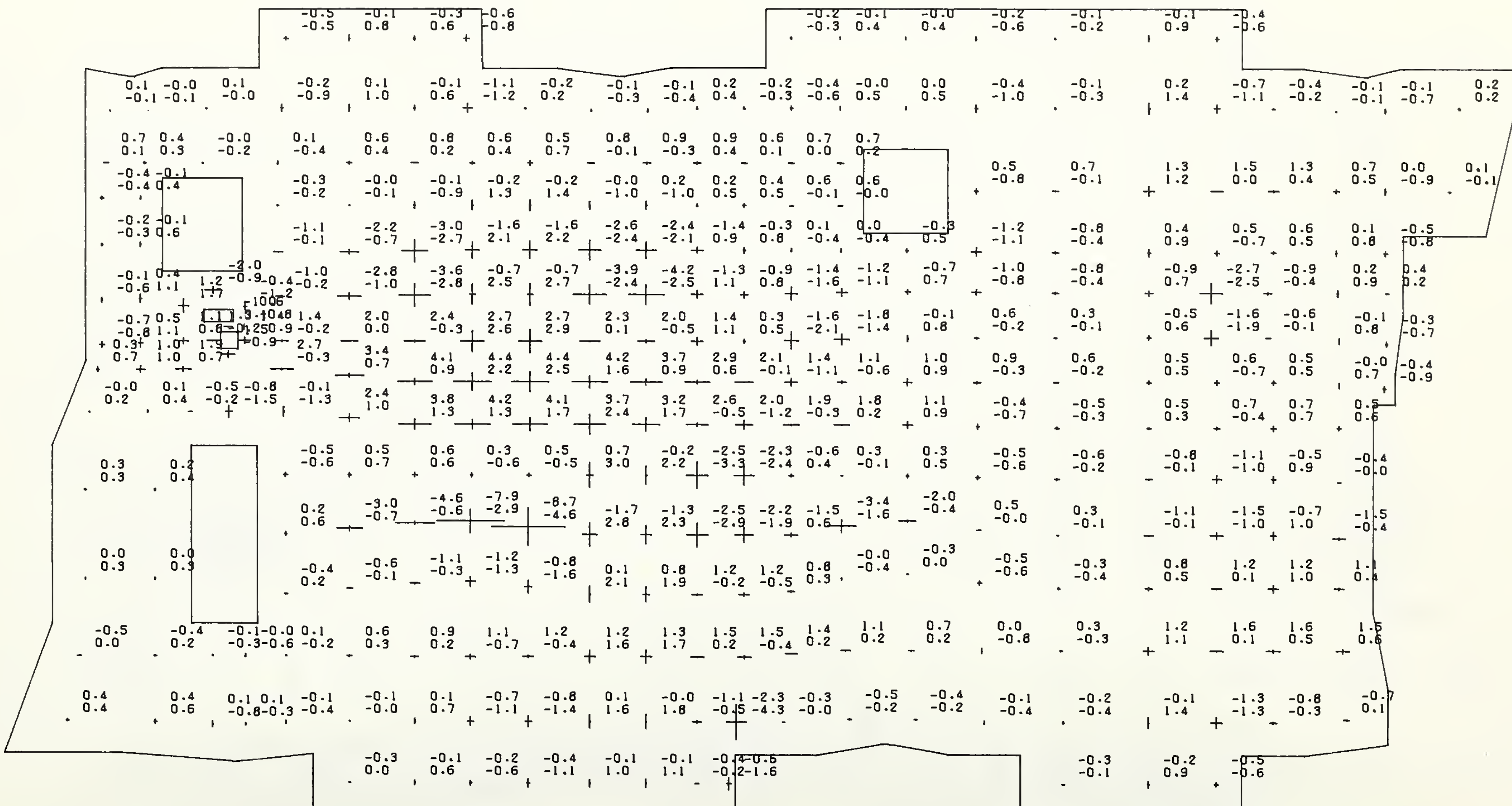


Figure 4.5 INTERNAL MOMENTS IN SLAB-CASE 3

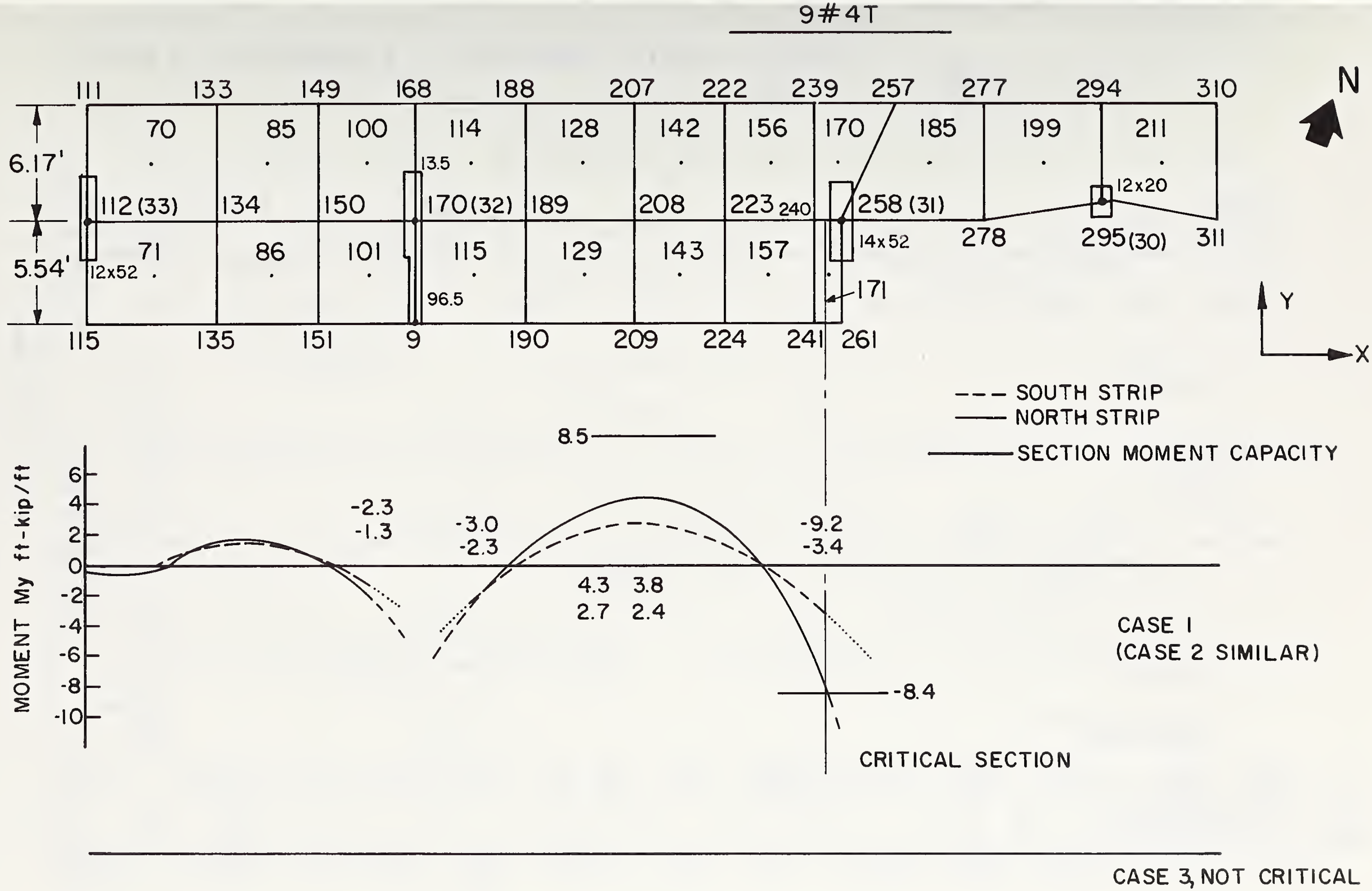


Figure 4.6 MOMENTS IN COLUMN STRIP 33-32-31-30

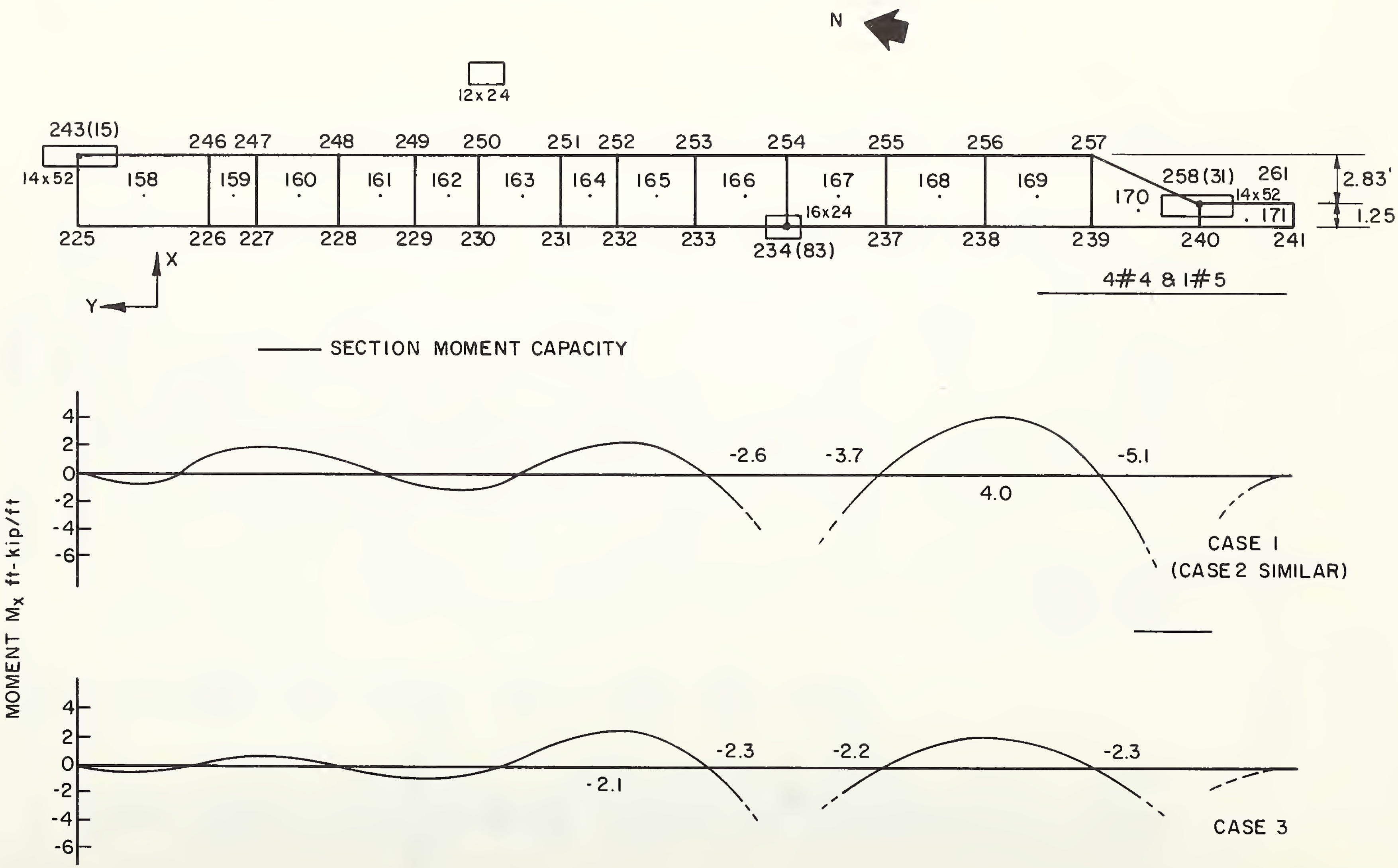


Figure 4.7 MOMENTS IN COLUMN STRIP 15-83-31

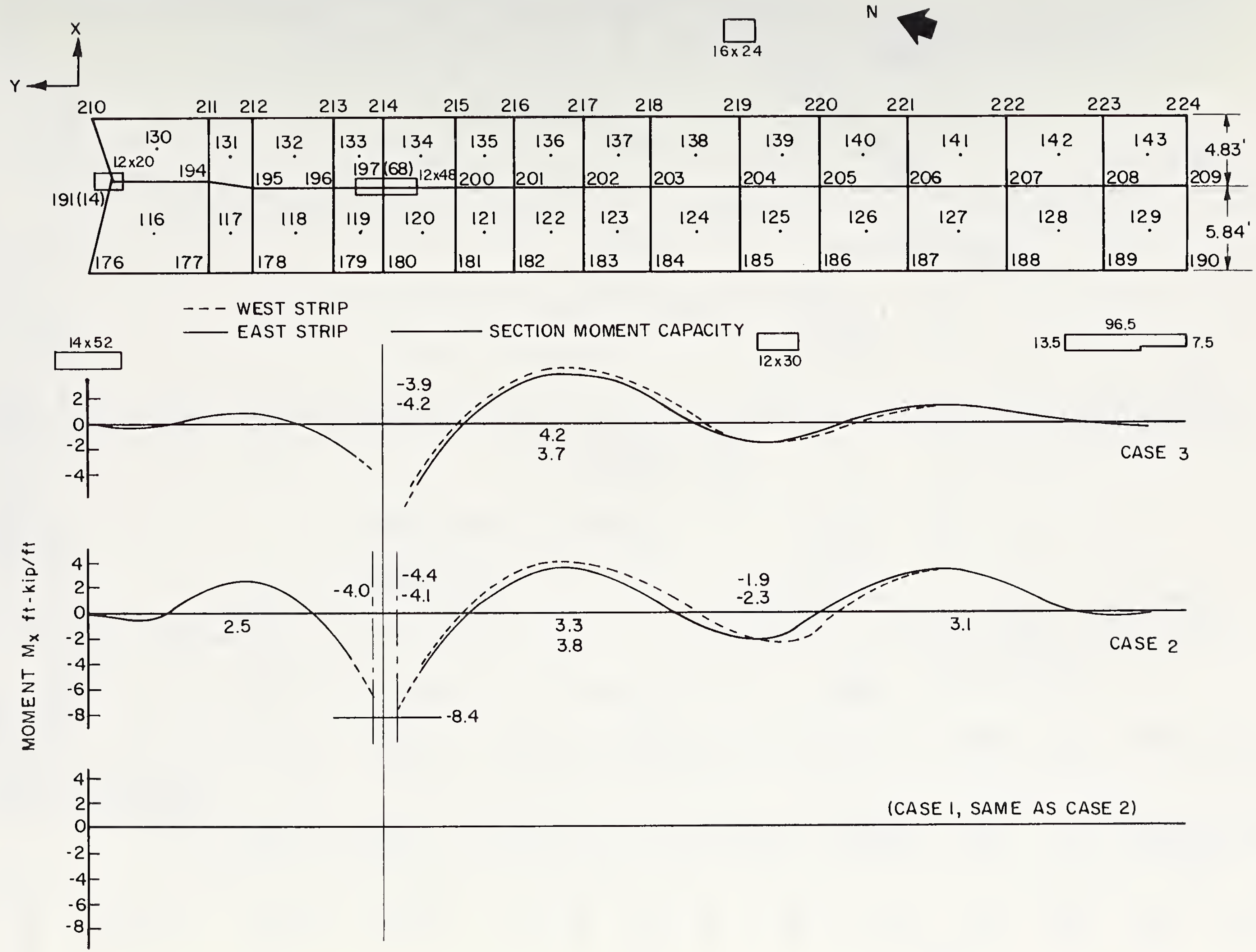


Figure 4.8 MOMENTS IN COLUMN STRIP 191-197

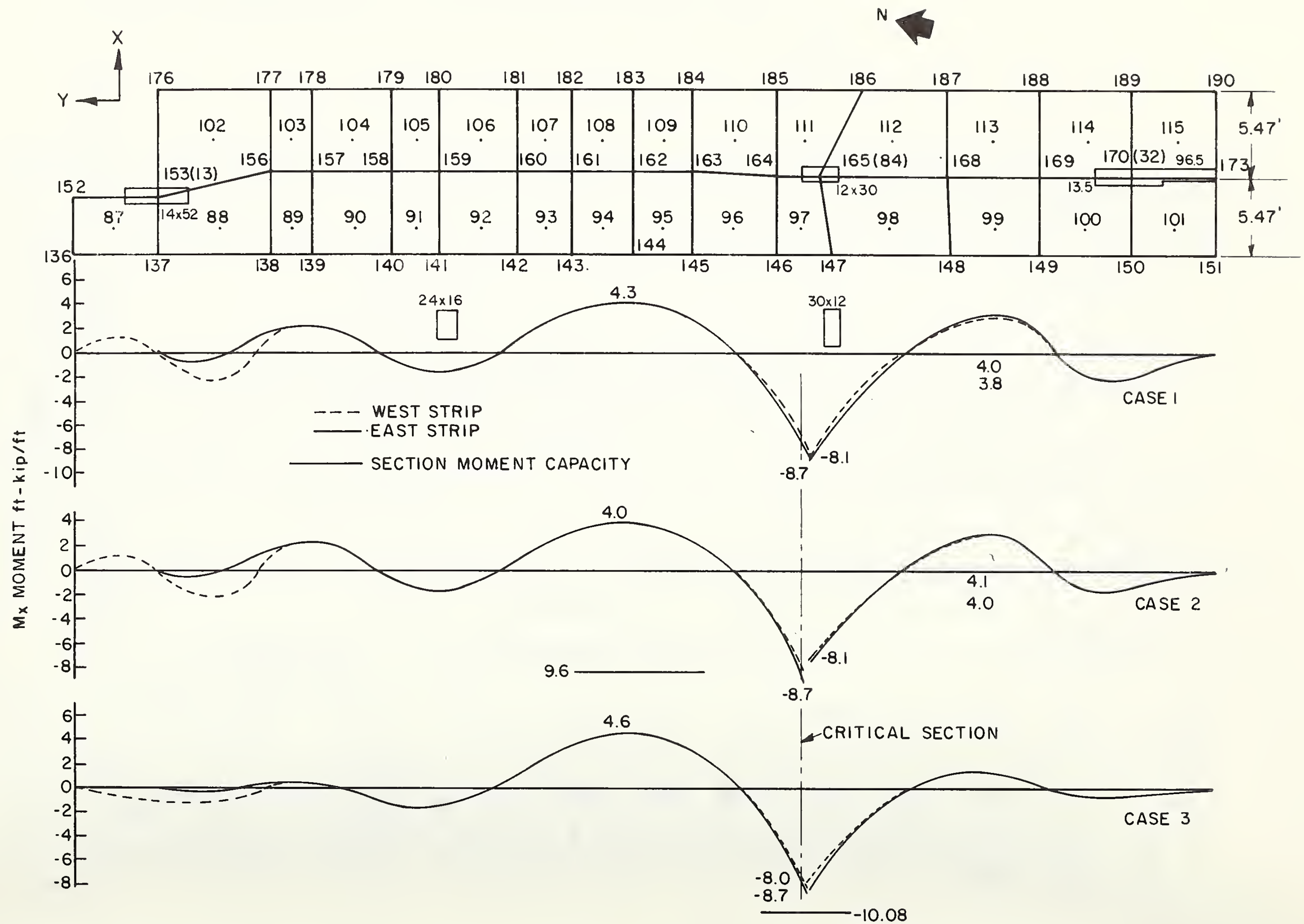


Figure 4.9 MOMENTS IN COLUMN STRIP 13-84-32

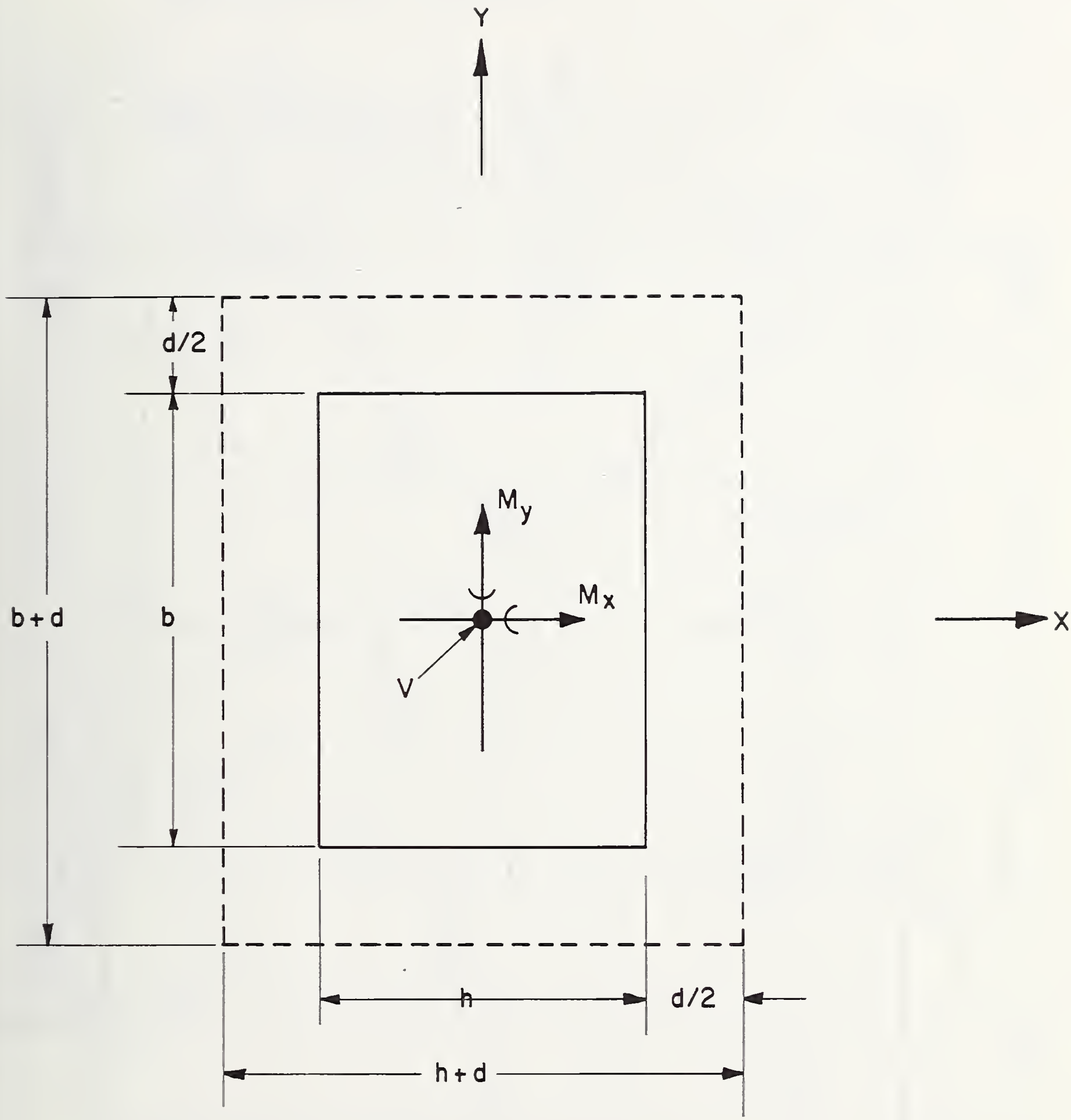


Figure 4.10 TYPICAL RECTANGULAR COLUMN SHOWING CRITICAL SECTION FOR SHEAR

SCALE: 1/4" = 1'-0"

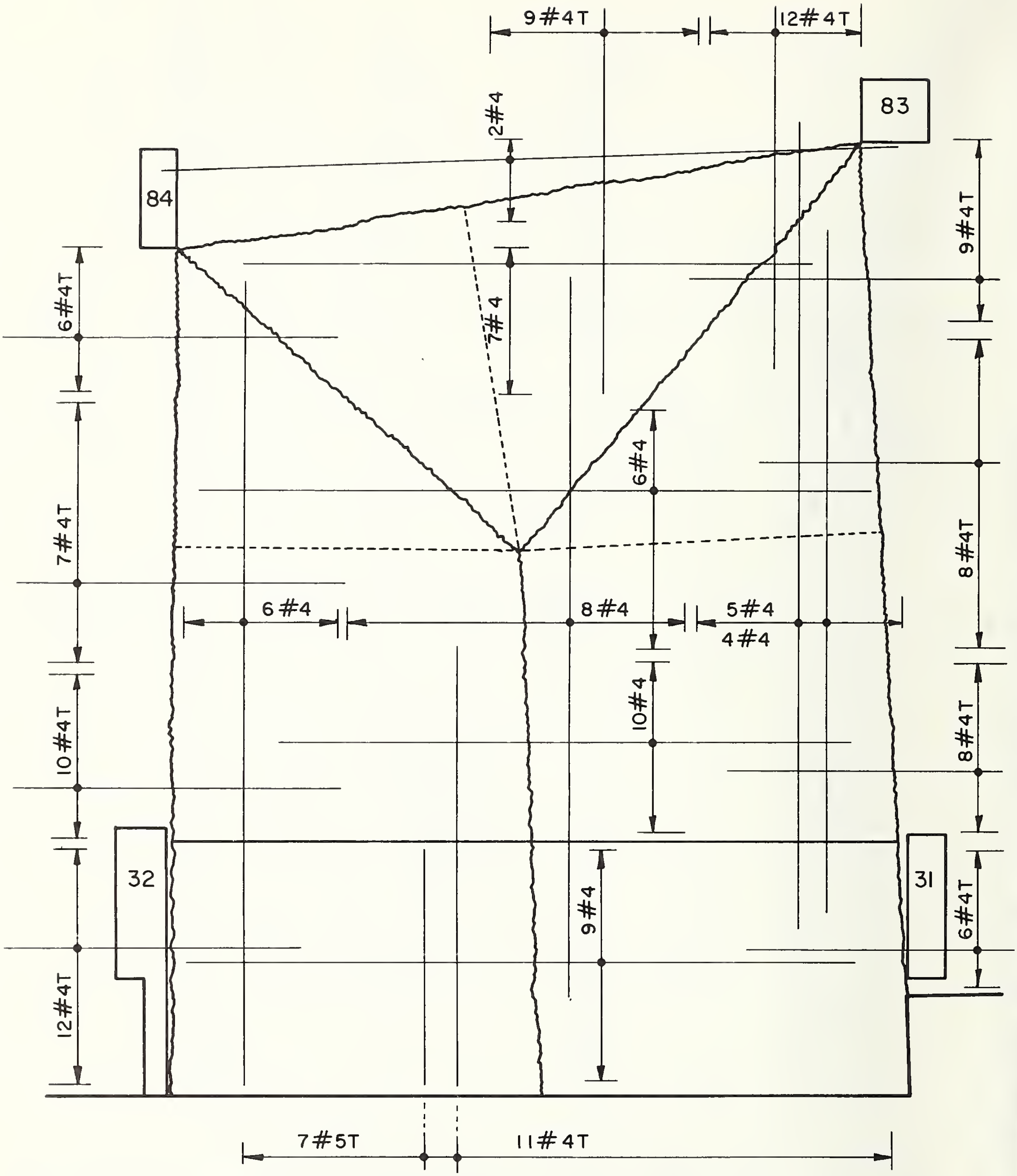


Figure 4.11 YIELDLINE PATTERN IN BALCONY PANEL

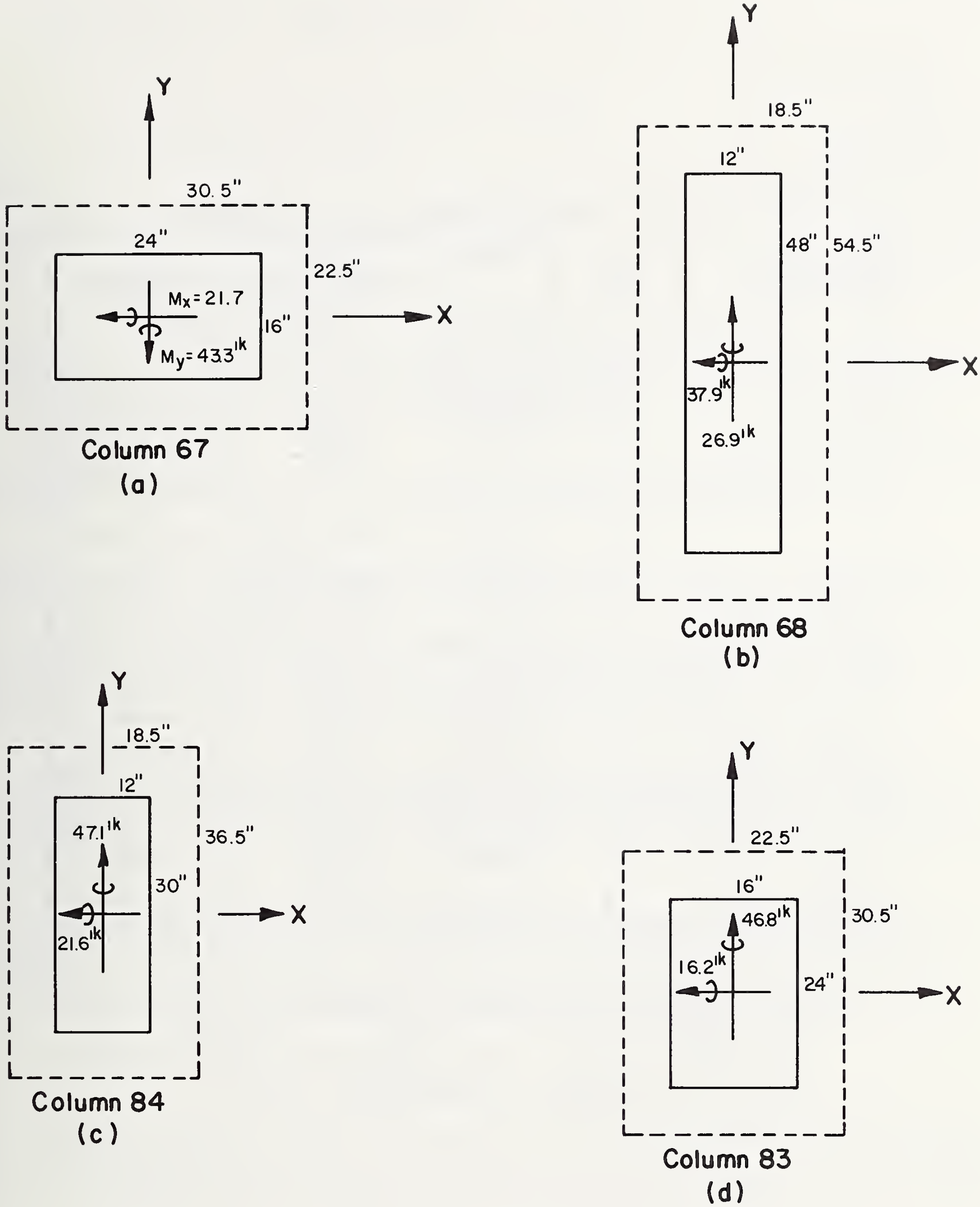


Figure 4.12 CRITICAL SECTION FOR SHEAR IN THE SLAB AROUND COLUMNS 67, 68, 83, 84

Column No.	Case	v_{max} psi	f'_c [1/] required psi	f'_c estimated psi
	1	147	1870	960-1440
67	2	147	1870	3000
(16 x 24)	3	128	1417	960-1440
	1	113	1104(1724)	960-1440
68	2	113	1104(1724)	3000
(48 x 12)	3	116	1164(1816)	960-1440
	1	148	1894	960-1440
83	2	148	1894	3000
(24 x 16)	3	130	1462	960-1440
	1	149	1920	960-1440
84	2	149	1920	3000
(30 x 12)	3	145	1819	960-1440

[1/] The values in brackets are based on $v_{max} > 0.8 (3.4\sqrt{f'_c})$ or $f'_c = 0.135\ v_{max}^2$ to account for the effect of column rectangularity [13A].

Figure 4.13 Summary of shear stresses in the 23rd story floor slab.

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15. SUPPLEMENTARY NOTES

16. ABSTRACT *(A 200-word or less factual summary of most significant information. If document includes a significant bibliography or literature survey, mention it here.)*

The collapse of the Skyline Plaza apartment building A-4 has been studied by using information contained in case records of the Occupational Safety and Health Administration (OSHA), U. S. Department of Labor and obtained from on-site inspections by investigators from the National Bureau of Standards.

Non-compliance with OSHA construction standards has been identified with regard to formwork, field-cured concrete specimens and crane installation. Specifically, the construction procedures did not comply with standards for the removal of supporting forms. It is concluded that premature removal of forms was a contributing factor to the collapse in building A-4.

An analysis of the 23rd-floor slab indicates that its most likely mode of failure was in shear around one or more columns in section 3 of the floor slab. The strength of the 23rd-floor slab on the day of collapse has been estimated to be at a level that removal of shoring could have produced shear failure in the slab.

17. KEY WORDS *(six to twelve entries; alphabetical order; capitalize only the first letter of the first key word unless a proper name; separated by semicolons)*

Apartment building; collapse; concrete; concrete strength; flexure; shear; strength

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