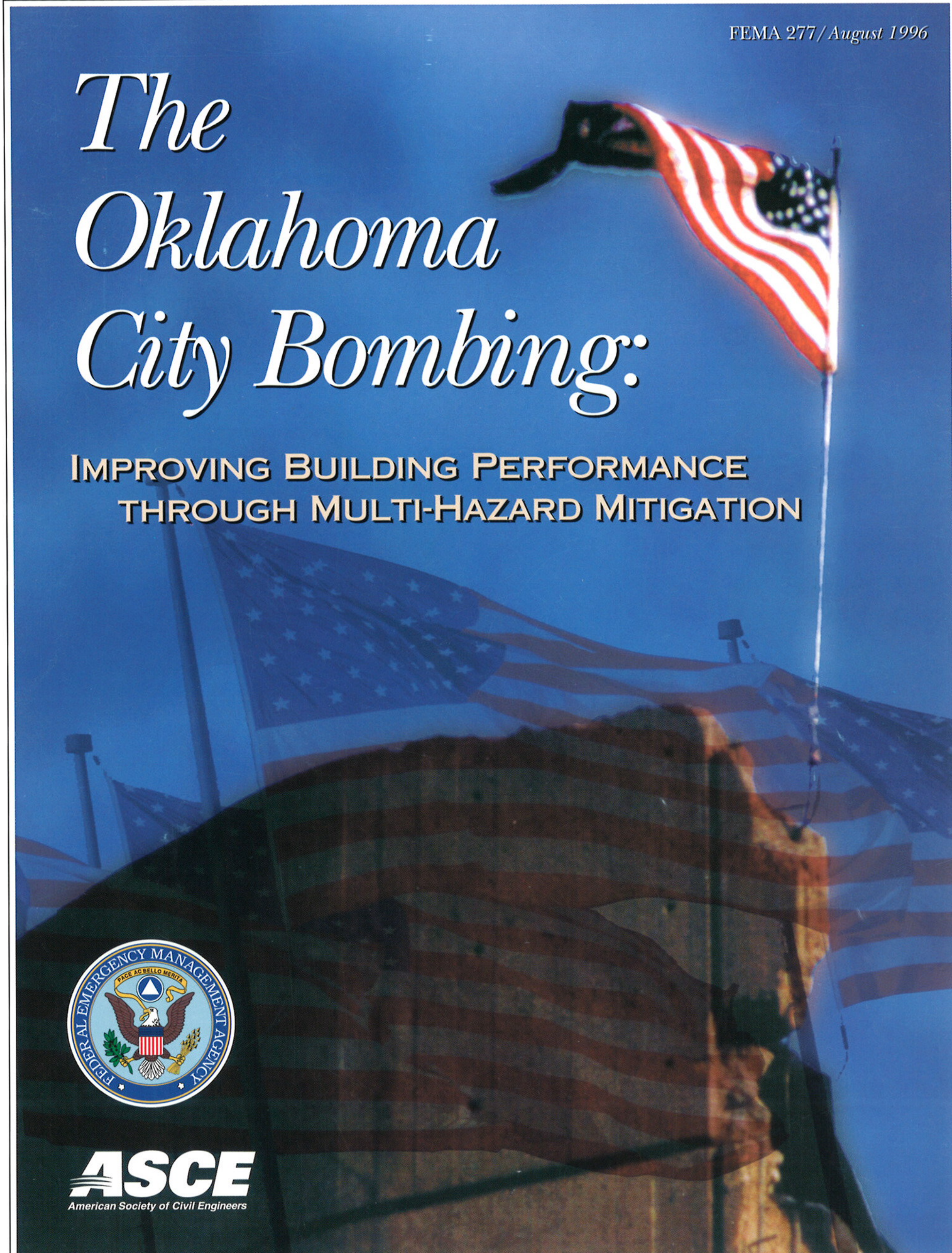
FEMA 277/*August 1996*

The Oklahoma City Bombing:

IMPROVING BUILDING PERFORMANCE THROUGH MULTI-HAZARD MITIGATION



FEDERAL EMERGENCY MANAGEMENT AGENCY

ASCE

American Society of Civil Engineers

The Building Performance Assessment Team Process

In response to hurricanes, flood, earthquakes, and other disasters, the Federal Emergency Management Agency (FEMA) often deploys Building Performance Assessment Teams (BPATs) to conduct field investigations at disaster sites. The members of a BPAT include representatives of public sector and private sector entities who are experts in specific technical fields such as structural and civil engineering, building design and construction, and building code development and enforcement. BPATs inspect disaster-induced damages incurred by residential and commercial buildings and other manmade structures; evaluate local design practices, construction methods and materials, building codes, and building inspection and code enforcement processes; and make recommendations regarding design, construction, and code issues. With the goal of reducing the damage caused by future disasters, the BPAT process is an important part of FEMA's hazard mitigation activities.

August 30, 1996

The Oklahoma City Bombing:

IMPROVING BUILDING PERFORMANCE THROUGH MULTI-HAZARD MITIGATION

FEDERAL EMERGENCY MANAGEMENT AGENCY
MITIGATION DIRECTORATE

ASCE
American Society of Civil Engineers

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Executive Summary

In May 1995, the Federal Emergency Management Agency (FEMA) deployed a Building Performance Assessment Team (BPAT) composed of American Society of Civil Engineers (ASCE) and Federal Government engineers to investigate damage caused by the malevolent bombing of the Alfred P. Murrah Federal Building.

The purposes of the investigation were to review damage caused by the blast, determine the failure mechanism for the building, and review engineering strategies for reducing such damage to new and existing buildings in the future. Specifically, mechanisms for multi-hazard mitigation, including mitigation of wind and earthquakes effects, were considered. Among the strategies evaluated were procedures and details provided in FEMA's 1994 Edition of *NEHRP (National Earthquake Hazards Reduction Program) Recommended Provisions for Seismic Regulations for New Buildings* (Reference 7).

The BPAT visited the area around the Murrah Building in Oklahoma City during the period of May 9 through 13, 1995, 3 weeks after the blast occurred on Wednesday, April 19, 1995. The location of the Murrah Building in downtown Oklahoma City is shown in Figures 1-2 and 1-3 in Section 1.

While in Oklahoma City, the BPAT took photographs; collected structural drawings, shop drawings, photographs, and samples of structural components, including concrete and reinforcing bars; and obtained an audio tape of the explosion. The BPAT also conducted interviews concerning damage to buildings. Physical inspection of the structure was limited to visual observation from a distance of approximately 200 feet.

From visual inspection and analysis of the damage that occurred in the Murrah Building as a result of a blast caused by a truck bomb, it is concluded that progressive collapse extended the damage somewhat beyond that caused directly by the blast. The type of damage that occurred and the resulting collapse of nearly half the building is consistent with what would be expected for an Ordinary Moment Frame building of the type and detailing available in the mid-1970's when subjected to the blast from the large bomb that was detonated.

Using information developed for FEMA and the Department of Housing and Urban Development, it is possible to identify types of structural systems that would provide significant increases in toughness to structures subjected to catastrophic loading from events such as major earthquakes and blasts. One of these systems is compartmentalized construction, in which a large percentage of the building has structural walls that are reinforced to provide structural integrity in case the building is damaged. Two additional types of detailing, used in areas of high seismicity, are Special Moment Frame construction and Dual Systems with Special Moment Frames (herein referred to as Dual Systems).

The compartmentalized type of construction has been proven to provide significant structural integrity and resistance to progressive collapse as shown in Figure 6-1 in Section 6. However, the small and inflexible spaces created by this type of construction are not well-suited to office buildings. For this reason, compartmentalized construction could not often be used in Federal buildings.

Special Moment Frames and Dual Systems are frequently used in areas of high seismic activity. In this type of construction, ductile detailing (e.g., closed-hoop reinforcement to confine columns, continuous bars for continuity, and beam-to-column connections to transfer forces through the joints) provides toughness to resist blast and earthquake forces. ("Detailing" is the process of selecting and designating on drawings the amounts, lengths, bends, and locations of steel reinforcement in reinforced concrete.) Special Moment Frames and Dual Systems are described in *NEHRP (National Earthquake Hazards Reduction Program) Recommended Provisions for Seismic Regulations for New Buildings* (Reference 7) and Chapter 21 of the American Concrete Institute publication *Building Code Requirements for Reinforced Concrete* (Reference 3).

Structural members reinforced as Special Moment Frames can provide better resistance to progressive collapse than can Ordinary Moment Frames such as used in the Murrah Building. Special Moment Frames and Dual Systems can provide very large open spaces. Consequently, they are suitable for construction of Federal office buildings.

It should be noted that the most important aspect of using Special Moment Frames or Dual Systems is the ductility detailing. The performance of the Murrah Building suggests that no increase in lateral load resistance is needed. Rather, different detailing is needed to provide structural integrity following damage caused by a bomb. The necessary detailing can be provided through the application of procedures recommended for earthquake resistance.

Compartmentalized construction, Special Moment Frames, and Dual Systems provide the mass and toughness necessary to reduce the effects of extreme overloads on buildings. Consequently, it is recommended that these structural systems be considered where a significant risk of seismic and/or blast damage exists.

Although it is not possible to prevent severe damage in the immediate area where the blast occurs, several strategies can be used to reduce potential damage to existing buildings. Among the strategies considered are rehabilitation/retrofitting of buildings and increasing standoff distances (i.e., the distance from the building to locations where vehicle bombs can be placed). When building use permits, providing increased standoff distance is cost-effective.

Investigations to determine the cost of using Special Moment Frames rather than Ordinary Moment Frames were conducted by the Building Seismic Safety Council. These investigations, along with more recent changes in designs, suggest that the average increase in cost is in the range of 1 to 2 percent of the total cost of the building. This additional cost can be expected to be within the normal differences between high and low construction bids on a Federal building.

From these observations, it appears desirable and economically feasible to use compartmentalized construction, Special Moment Frame designs, or Dual Systems for Federal buildings where a significant risk of seismic and/or bomb damage exists.

When the benefits and costs associated with either seismic- or blast-resistant designs are being analyzed, the additional benefits derived from providing tangential blast resistance, in the case of seismic designs, and tangential seismic resistance, in the case of blast designs, should be considered. The inclusion of such additional benefits may provide economic justification for designs that offer greater toughness.

In combination with strategies reviewed in Section 5 of this report, application of mitigation strategies developed for FEMA for wind and earthquake can significantly improve blast resistance.

1 Introduction

1.1 Purpose and Scope

1.1.1 Purpose

The Federal Emergency Management Agency (FEMA) deployed a Building Performance Assessment Team (BPAT) composed of American Society of Civil Engineers (ASCE) and Federal Government engineers to investigate damage caused by the malevolent bombing of the Alfred P. Murrah Federal Building (hereafter referred to as the "Murrah Building") in Oklahoma City, Oklahoma (Figure 1-1). The BPAT included four ASCE team members who are experts in forensic engineering, behavior of structures under catastrophic loadings, and physical security of structures. The team also included engineers from FEMA, the U.S. Army Corps of Engineers, the General Services Administration (GSA), and the National Institute of Standards and Technology (NIST). Refer to Appendix A for additional information regarding the BPAT members.

The purposes of the investigation were to review damage caused by the blast, determine the failure mechanism for the building, and review engineering strategies for reducing such damage to new and existing buildings in the future. Specifically, mechanisms for multi-hazard mitigation, including mitigation of wind and earthquakes effects, were considered. Among the strategies evaluated were procedures and details provided in FEMA's 1994 Edition of *NEHRP (National Earthquake Hazards Reduction Program) Recommended Provisions for Seismic Regulations for New Buildings* (Reference 7).

This report presents recommendations concerning the design and construction of new Federal buildings. Also provided are mitigation recommendations for existing Federal buildings.

1.1.2 Scope of Work

The BPAT visited the area around the Murrah Building in Oklahoma City during the period of May 9 through 13, 1995, 3 weeks after the blast occurred on Wednesday, April 19, 1995. The location of the Murrah Building in downtown Oklahoma City is shown in Figures 1-2 and 1-3.

While in Oklahoma City, the BPAT took photographs; collected structural drawings, shop drawings, photographs, and samples of structural components, including concrete and reinforcing bars; and obtained an audio tape of the blast. The BPAT also conducted interviews concerning damage to buildings. Physical inspection of the structure was limited to visual observation from a distance of approximately 200 feet.

Upon completion of the site visit, each team member was assigned a portion of the analytical work. Samples of concrete and reinforcing bars taken from the site were tested to determine physical properties of materials used in the building. The work performed included developing the most probable response of the building to the blast and determining whether new technology can be used to enhance the resistance of buildings to blast, wind, earthquake, and other hazards.

Figure 1-1 *Murrah Building prior to blast.*

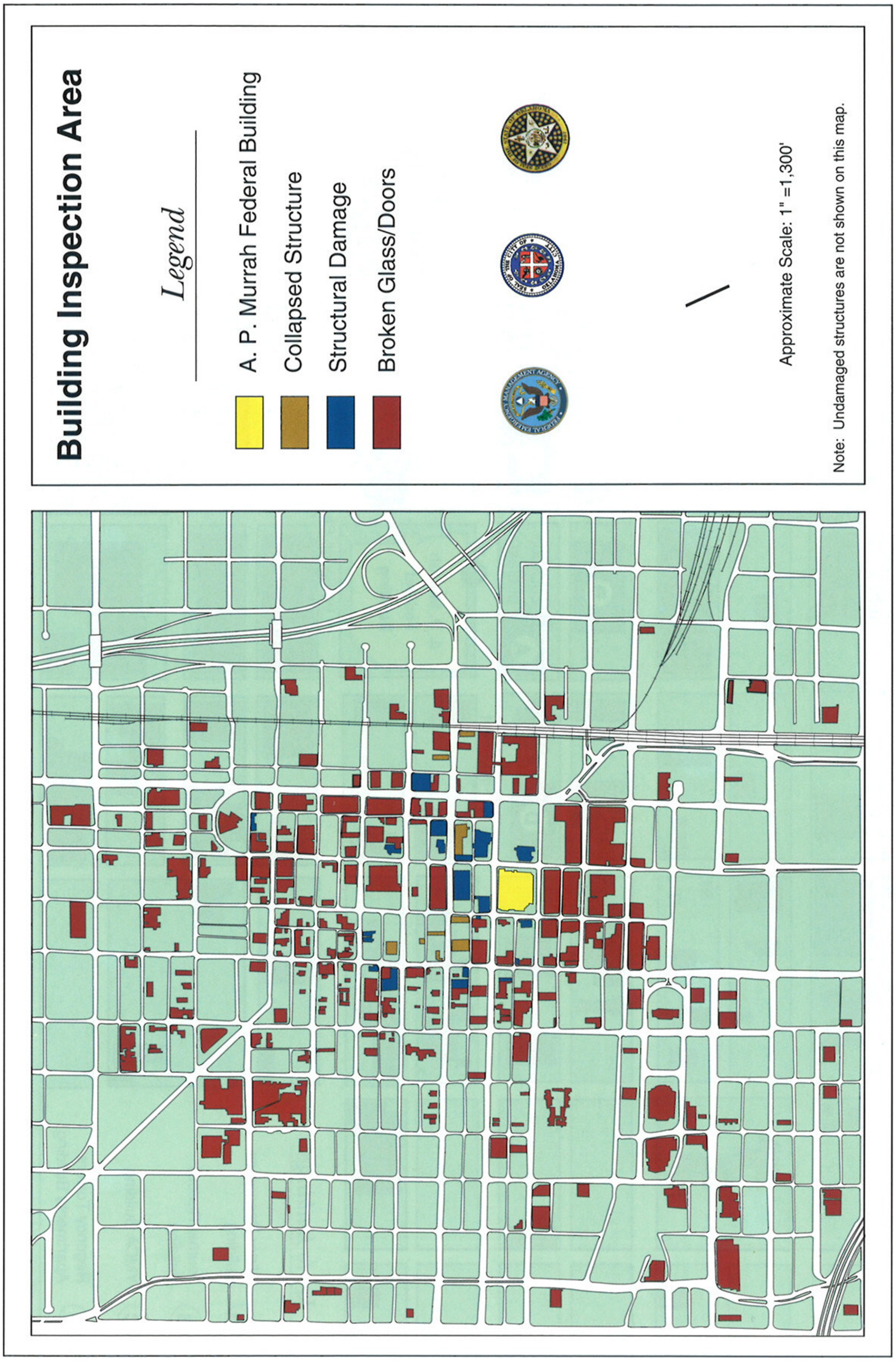


Figure 1-2 *Locations of Murrah Building and other damaged structures.*

Figure 1-3 Immediate vicinity of Murrah Building.

1.2 ALFRED P. MURRAH FEDERAL BUILDING

The Murrah Building project was designed for the Design & Construction Division, Region 7, Fort Worth, Texas, of the GSA Public Buildings Service, Washington, DC. The architect for the project was a joint venture of two firms, Shaw Associates and Locke Wright Foster, both based in Oklahoma City, Oklahoma. The individual architects of record were Wendel V. Locke (Registered Architect in Oklahoma, License No. 614) and William M. Shaw (Registered Architect in Oklahoma, License No. 620). The structural engineer of record for the project was Paul Ed Kirkpatrick (Registered Professional Engineer in Oklahoma, License No. 7597).

The contract for the design of the project was signed in the early 1970's, and the contract drawings that were issued for construction were dated May 6, 1974. Construction documents for the project consist of architectural, structural, mechanical, and electrical drawings, plus specifications for construction. The general contractor for the project was J. W. Bateson, Inc. Shop drawings for reinforcing bars were prepared for/by The Ceco Company between December 1974 and May 1975. A spot check of the Ceco reinforcing bar shop drawings shows compliance and good correlation with the structural contract documents. Construction was accomplished in 20 months between late 1974 and early 1976.

A tower crane was used in the construction of the Murrah Building. Provision was made for a 7-foot by 7-foot opening in all floors. Ceco shop drawings show this opening to be located midway between Column Lines 18 and 20, centered on Column Line F. (See Figure 1-4 for the site plan, which shows the column lines.) Photographs of the damaged building confirm this location. To compensate for the interrupted main reinforcing bars in the slab, the contractor installed additional reinforcing around the opening.

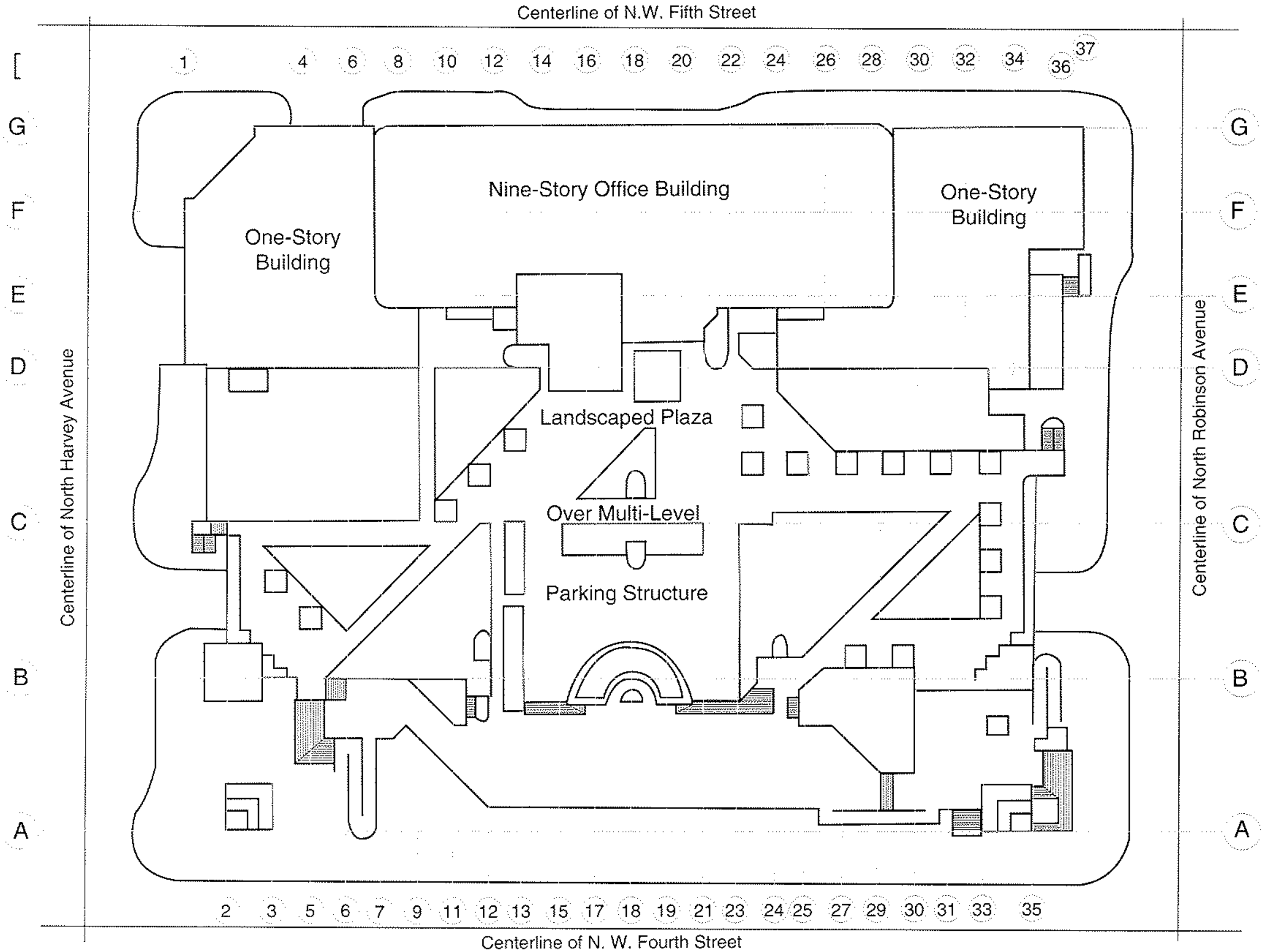
The Murrah Building project consists of a nine-story office building (hereafter referred to as the nine-story portion of the Murrah Building) with one-story ancillary east and west wings and an adjacent multi-level parking structure, partially below grade and partially above grade, south of the office building.

As shown in Figure 1-4, the entire project is located between North Harvey Avenue on the west, N.W. Fifth Street on the north, North Robinson Avenue on the east, and N.W. Fourth Street on the south, between grid lines 1 and 37, which are numbered west to east, and grid lines A and G, which are lettered south to north.

The multi-level parking structure extends from grid line 2 to grid line 35 and from grid line A to grid line D. Approximate plan dimensions are 330 feet in the east-west dimension and 195 feet in the north-south dimension. The spans in the north-south direction are 65 feet. There are three levels of parking between grid lines A and B, four levels between grid lines B and C, and three levels between grid lines C and D. Concrete bearing walls in the east-west direction support the 65-foot-span precast double T-beams used in the parking structure. Directly atop the parking structure is a landscaped plaza at the approximate elevation of the second floor of the office building, 1235 feet to 1240 feet above mean sea level (m.s.l.).

The focus of this report is on the nine-story portion of the Murrah Building, which incurred significant damage and partial collapse as a result of the April 19, 1995, bombing. The nine-story and one-story portions of the building have since been demolished. The parking structure mentioned above was barely damaged and has not been demolished.

The nine-story frame of the Murrah Building was an Ordinary Moment Frame supported on columns located between grid line 8 on the west, grid line 28 on the east, grid line D on the south, and grid line G on the north (see Figure 1-4). The overall plan dimension was



Note: This figure is based on architectural drawing 2-2 / 8, titled "Site Plan," dated 5/6/74, prepared by Shaw Associates/Locke Wright Foster for the General Services Administration.

Approximate Scale: 1" = 60'

Figure 1-4 *Site plan for Murrah Building.*

approximately 220 feet in the east-west direction and approximately 100 feet in the north-south direction. The elevation at grade at the first floor was approximately 1222.5 feet m.s.l.

The architectural plan for the nine-story portion consisted of ten 20-foot bays in the east-west direction and two 35-foot bays in the north-south direction plus shear walls and other localized columns and walls in the core area at the midpoint of the south side of the building. The architectural expression used exposed reinforced concrete with a vertical-board-formed finish. Four large, prominent, vertical circular tube columns, one at each of the four corners of the building, acted as air intake/exhaust shafts for the ventilation system. See Figures 1-5, 1-6, 1-7, 1-8, and 1-9 for floor and roof plans.

The north face of the nine-story portion consisted of a curtainwall system utilizing ¼-inch bronze tempered glass spandrels at each floor and 1-inch bronze insulating glass in a curtainwall system for the balance of the exterior wall. The module for the building and the curtainwall was 5 feet. Figures 1-1 and 1-10 show the north elevation.

An important feature of the structural system was a transfer girder at the third-floor level. The transfer girder supported intermediate columns, thereby providing 40-foot column spacing for the first two levels of the north side. In addition, the curtain wall was set back several feet, providing a cave-like space over the first two levels on the north.

The south face of the nine-story portion consisted of a combination of 1-inch bronze insulating glass in the curtainwall system with precast concrete and vertical-board-formed exposed concrete. Precast concrete at floor levels provided spandrels. Board-formed core walls/shear walls were exposed on the exterior.

The east and west faces of the nine-story portion were framed by the vertical circular ventilation shafts at the corners; exposed concrete vertical-board-formed spandrels were located between the shafts and at the top or parapet. The infill walls between spandrels were 3-inch granite stone panels backed by vertical steel studs and drywall. See Figures 1-1, 1-11, and 1-12.

The typical floor-to-floor height was 13 feet for the third through eighth floors. The floor height for the ninth floor was 14 feet. The entire nine-story portion was laid out on a 5-foot by 5-foot typical horizontal and vertical module for layout and ceiling. The roof was at elevation 1340.5 feet m.s.l. A bituminous membrane roof was supported on lightweight insulating roof on 1-inch vent board. Apparently, a monofilament membrane was added after the building had been in service for some time. Concrete roof pedestals were supplied for support of window washing equipment.

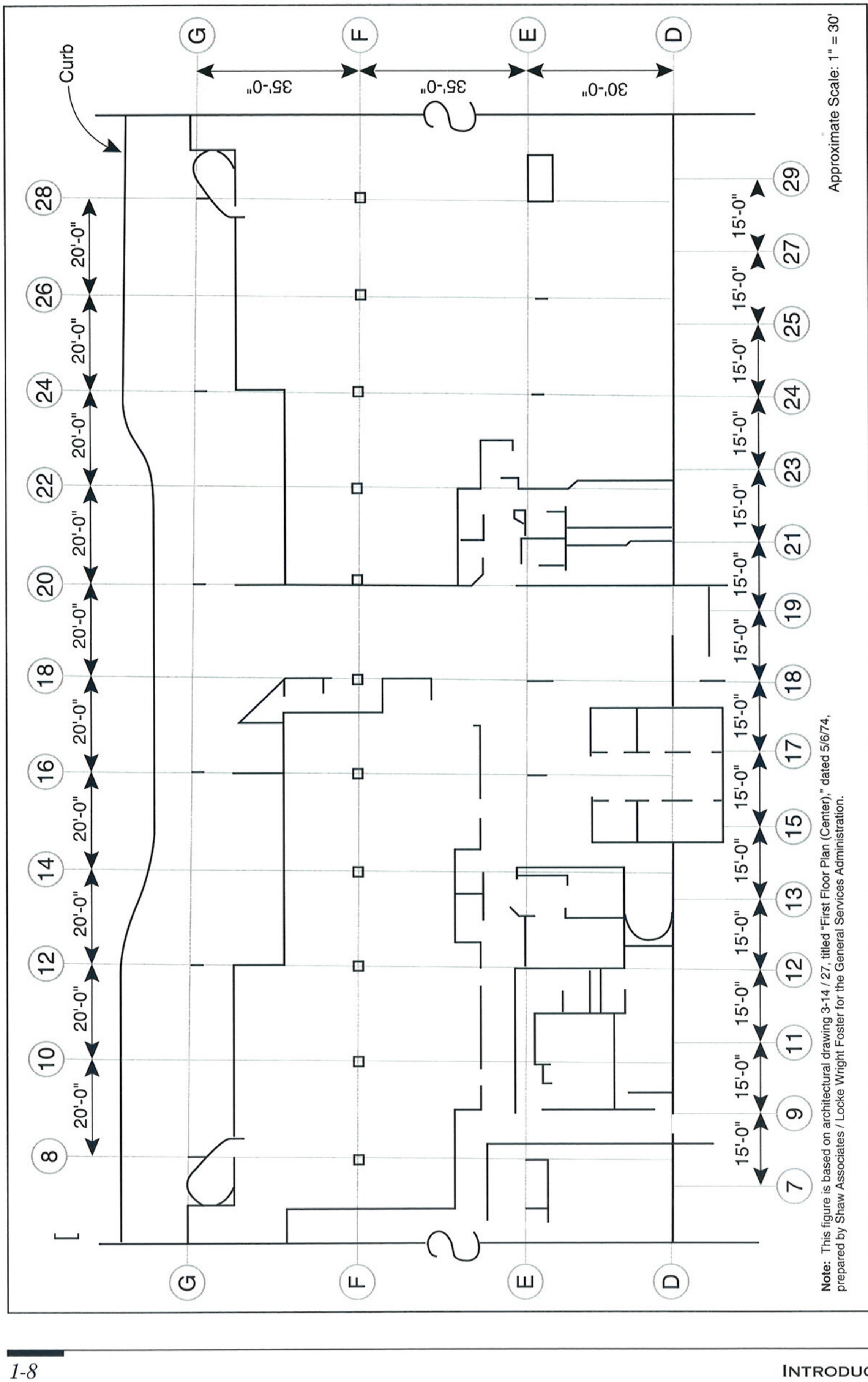
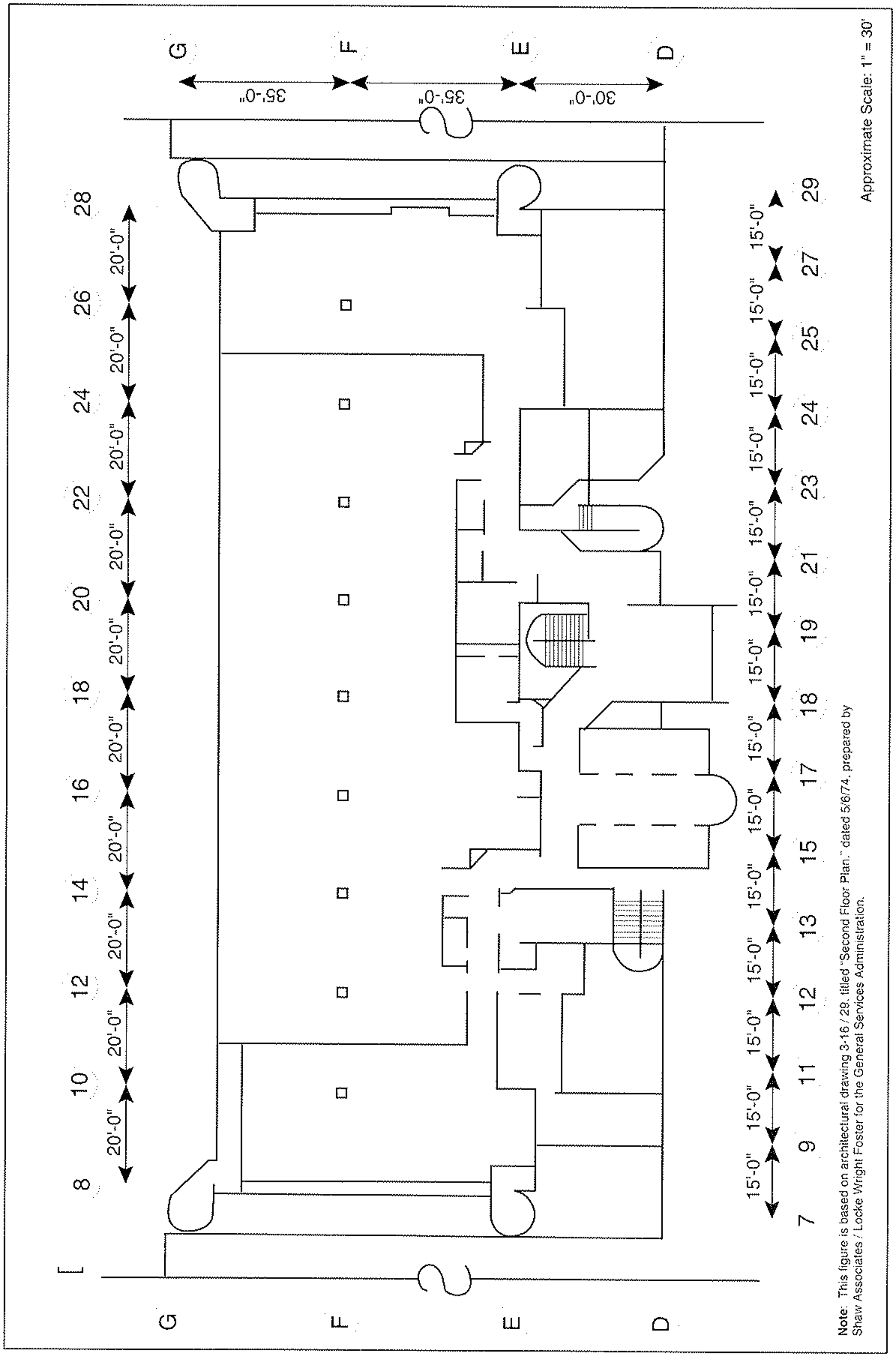
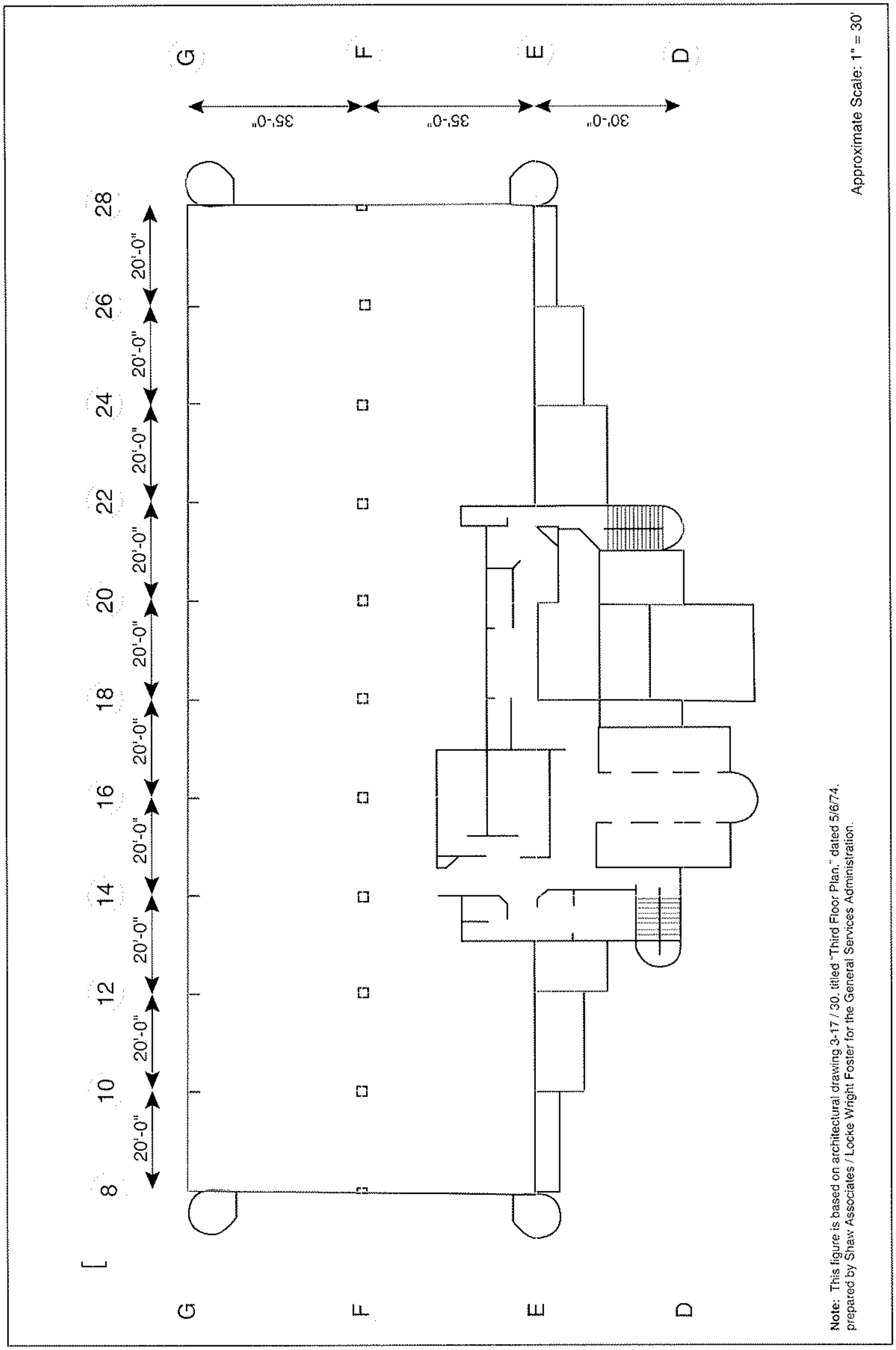


Figure 1-5 *Floor Plan – first floor of nine-story portion of Murrah Building.*



Note: This figure is based on architectural drawing 3-16 / 29, titled "Second Floor Plan," dated 5/6/74, prepared by Shaw Associates / Locke Wright Foster for the General Services Administration.

Figure 1-6 Floor plan – second floor of nine-story portion of Murrah Building.



Note: This figure is based on architectural drawing 3-17 / 30, titled "Third Floor Plan," dated 5/6/74, prepared by Shaw Associates / Locke Wright Foster for the General Services Administration.

Figure 1-7 Floor plan – third floor of nine-story portion of Murrah Building.

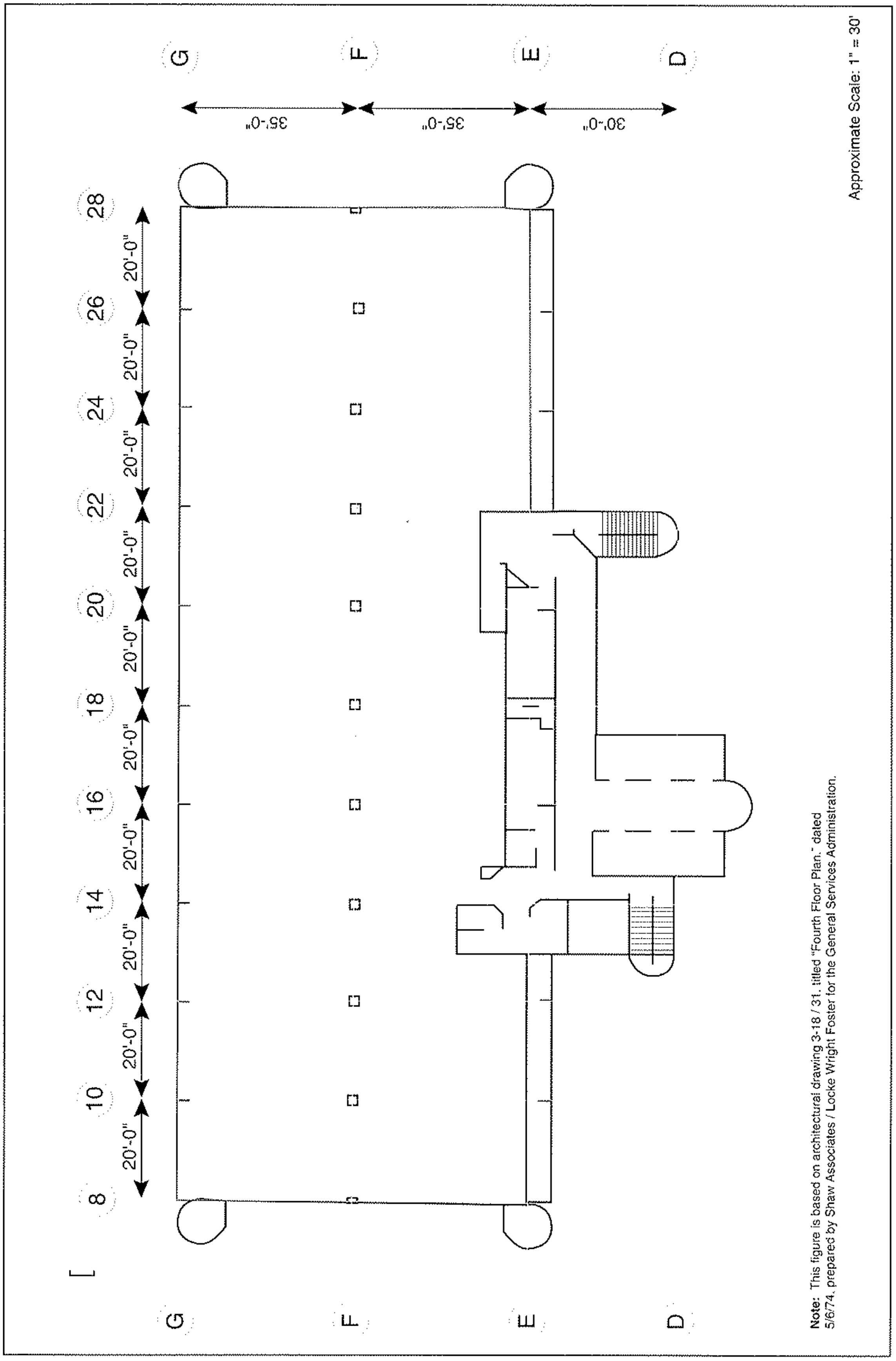
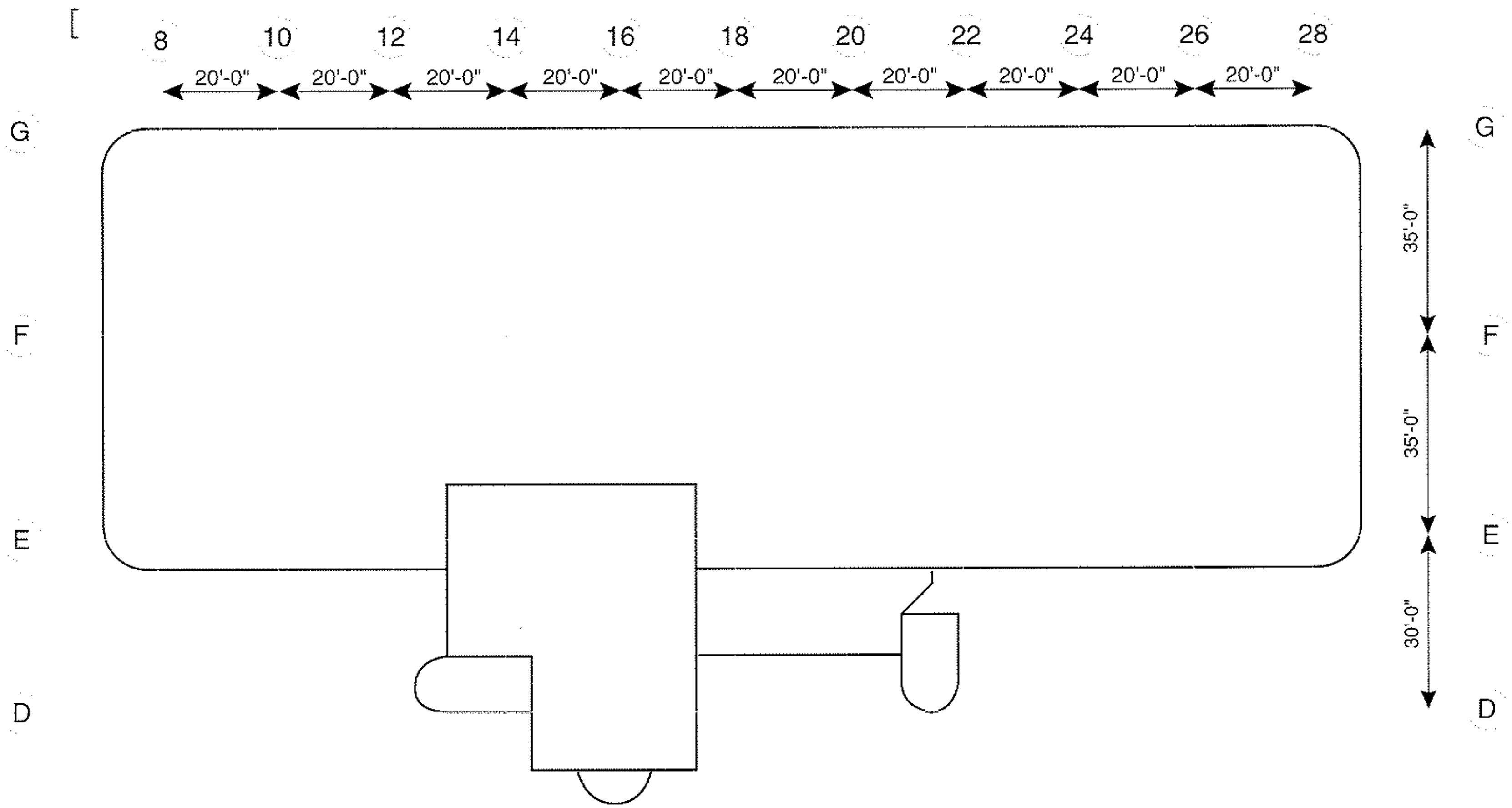


Figure 1-8 *Floor plan – fourth through ninth floors of nine-story portion of Murrah Building.*



Note: This figure is based on architectural drawing 3-25 / 38, titled "Roof Plan," dated 5/6/74, prepared by Shaw Associates / Locke Wright Foster for the General Services Administration.

Approximate Scale: 1" = 30'

Figure 1-9 *Roof plan – nine-story portion of Murrah Building.*

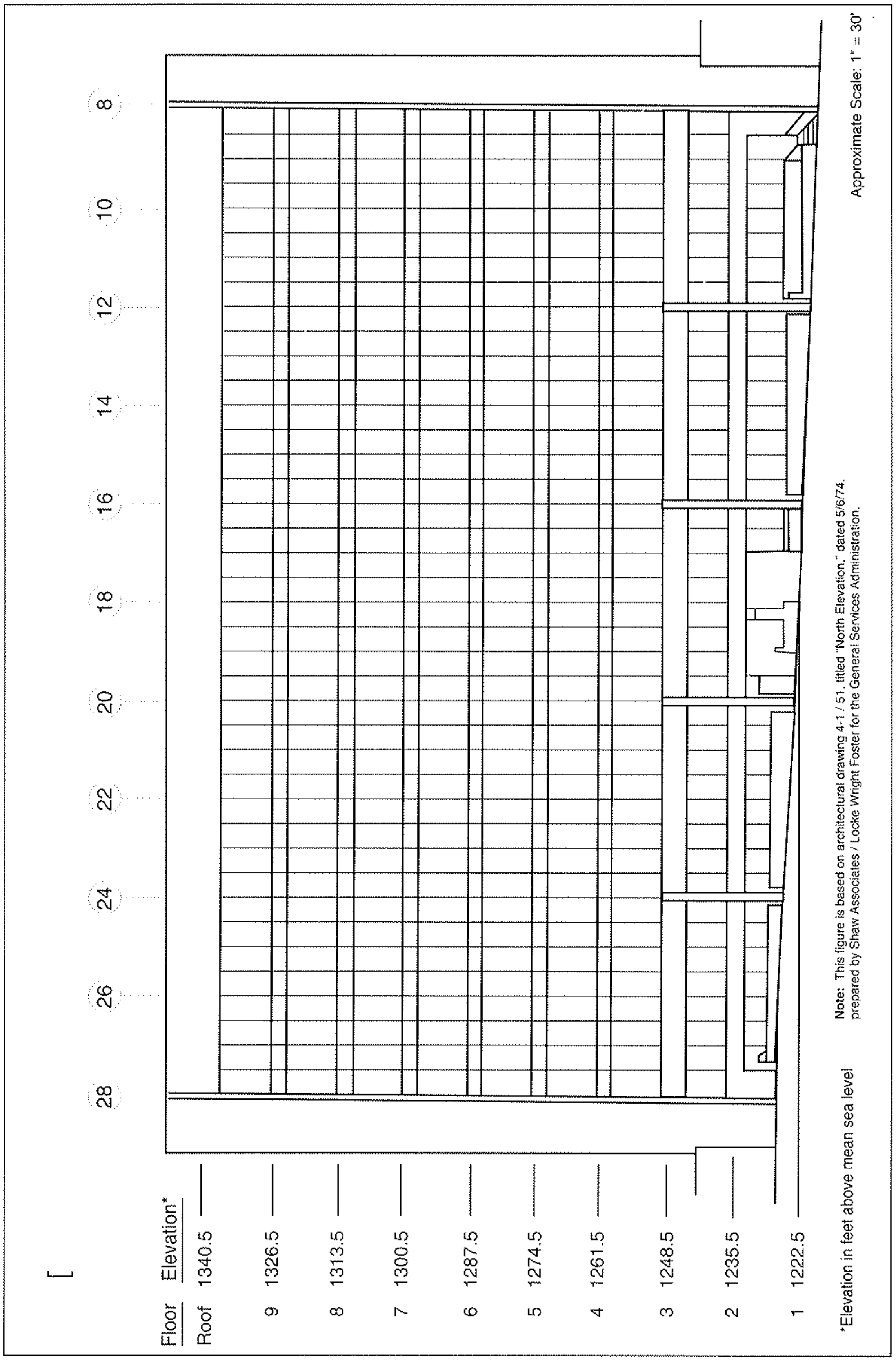


Figure 1-10 North elevation – nine-story portion of Murrah Building.

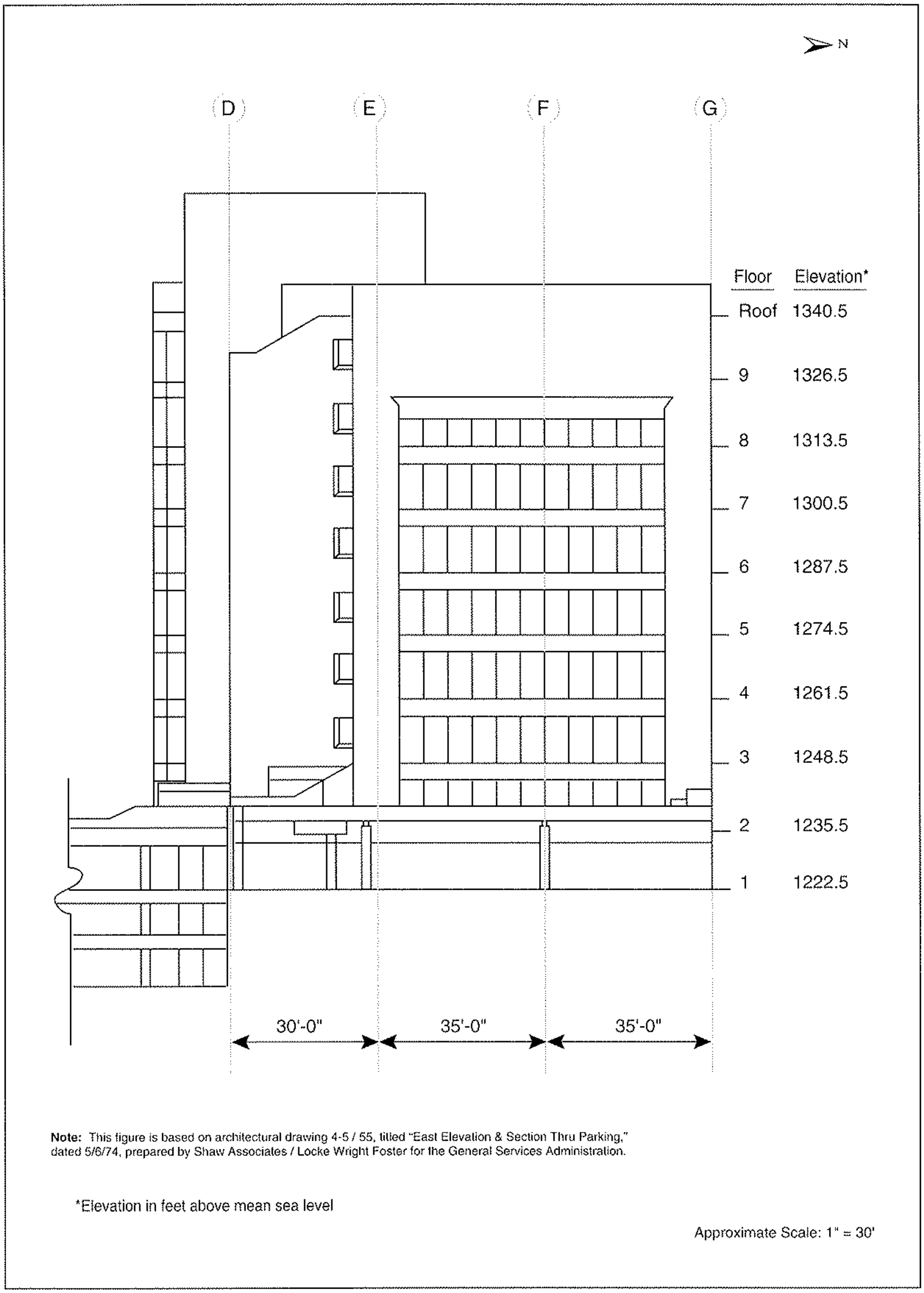


Figure 1-11 *East elevation – nine-story portion of Murrah Building.*

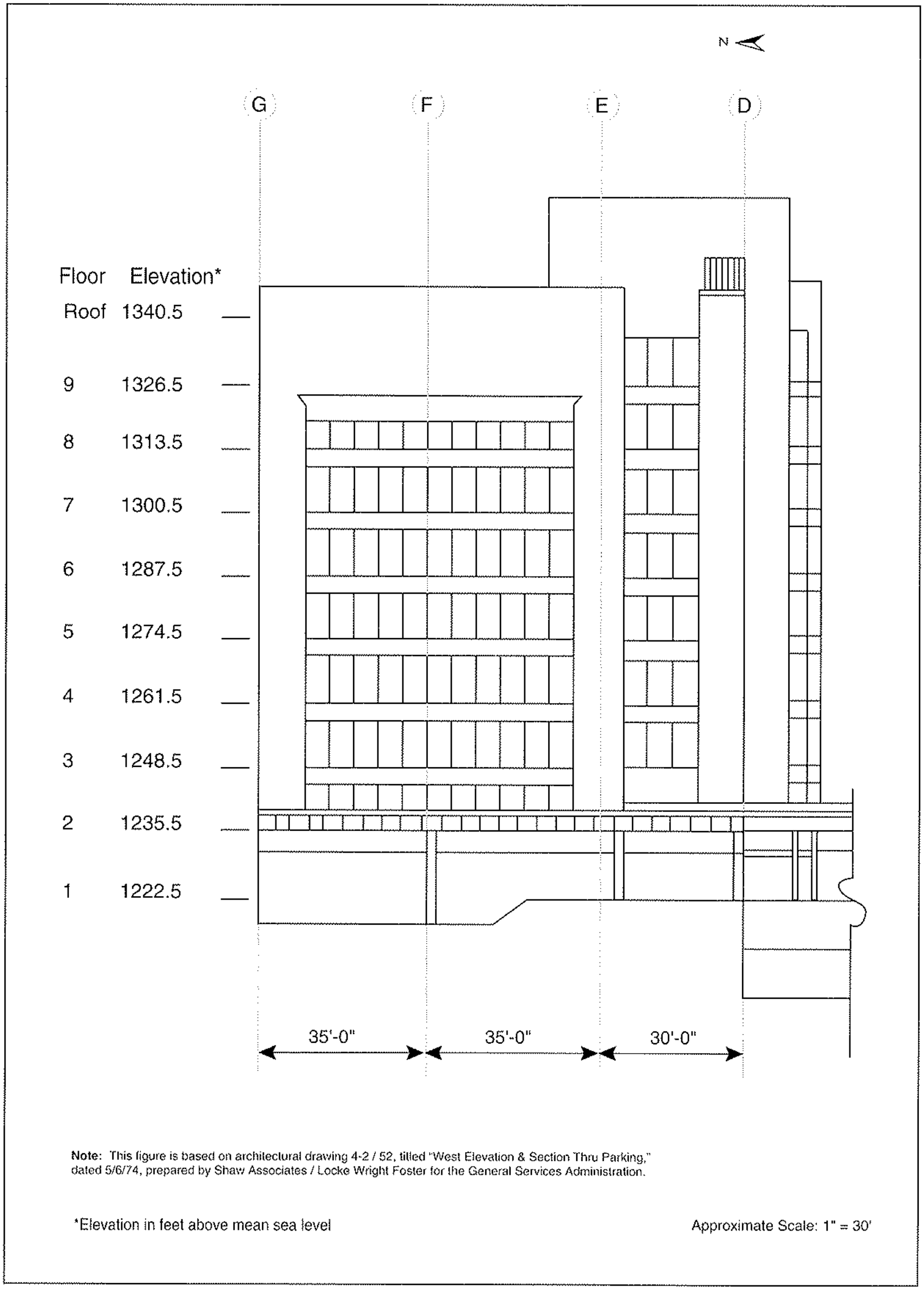


Figure 1-12 West elevation – nine-story portion of Murrah Building.

1.3 Response of Buildings to Blast

At 9:02 on the morning of April 19, 1995, a malevolent bomb blast occurred, causing major loss of life and injuries, significant structural damage, and partial collapse of the Murrah Building. Other buildings, primarily those nearby and in direct line-of-sight from the blast, suffered various degrees of blast damage (see Figures 1-2, 1-3, and 1-13). Located to the north and west of the nine-story portion of the Murrah Building is the State of Oklahoma Water Resources building (Water Resources building), due north is the Journal Record building, to the north and east is the YMCA building, and one block west of the Water Resources building is the Regency Towers Apartments building (see Figure 1-3).

1.3.1 Nine-Story Portion of Murrah Building

Major structural damage and building collapse occurred at the north side of the Murrah Building, which faced the blast (Figure 1-14). Here, most of the north half of the rectangular footprint, between Columns G12 and G28 (except for the extreme west end), extending 35 feet into the building, collapsed. Three columns (G16, G20, and G-24) supporting the third- level transfer girder were destroyed. The destruction of these columns triggered the progressive collapse of floors above. In addition, between Column Lines 20 and 24 (a length of 40 feet), the collapse extended the full 70-foot width of the building (to, but not through, the south wall). Roughly half of the occupiable space in the nine-story portion of the building collapsed.

While the north face of the building sustained the brunt of the effects of the blast, structural damage to the remaining exposures was limited, as illustrated in Figures 1-15 and 1-16.

Ray Blakeney, Director of Operations for the Oklahoma Medical Examiner's Office, has estimated that up to 90 percent of the fatalities were the result of crushing caused by falling debris.

1.3.2 Other Buildings

Water Resources Building

The Water Resources building, shown in Figure 1-17, is located close to the center of the blast. The exterior walls, roof slab, and interior of this building incurred significant damage. As can be seen in Figure 1-17, the windows were blown inward.

Glazing (glass, windows) such as that used on the Water Resources building is generally not capable of sustaining inward blast pressure, which engulfs the building and acts on all exposures. However, exterior masonry elements are more massive and can sustain a significant inward loading pulse. Consequently, masonry cladding may survive the inward blast loading. In some circumstances, as appears to be the case of the Water Resources building, the exterior skin is forced outward by a combination of rebound and the low-pressure (rarefaction) portion of blast pressure wave. The masonry connections to the building, or the masonry itself, failed in this outward motion (see Figure 1-18).

By far the most severe observable damage to the building was to the concrete roof slab. Here, the positive or downward loading of the blast pressure caused not only flexural cracks at the underside of the slab, but shear "punching" of the slab at interior columns. This is evident in the consistent spalling at the underside of the roof slab at interior columns (see Figure 1-19).

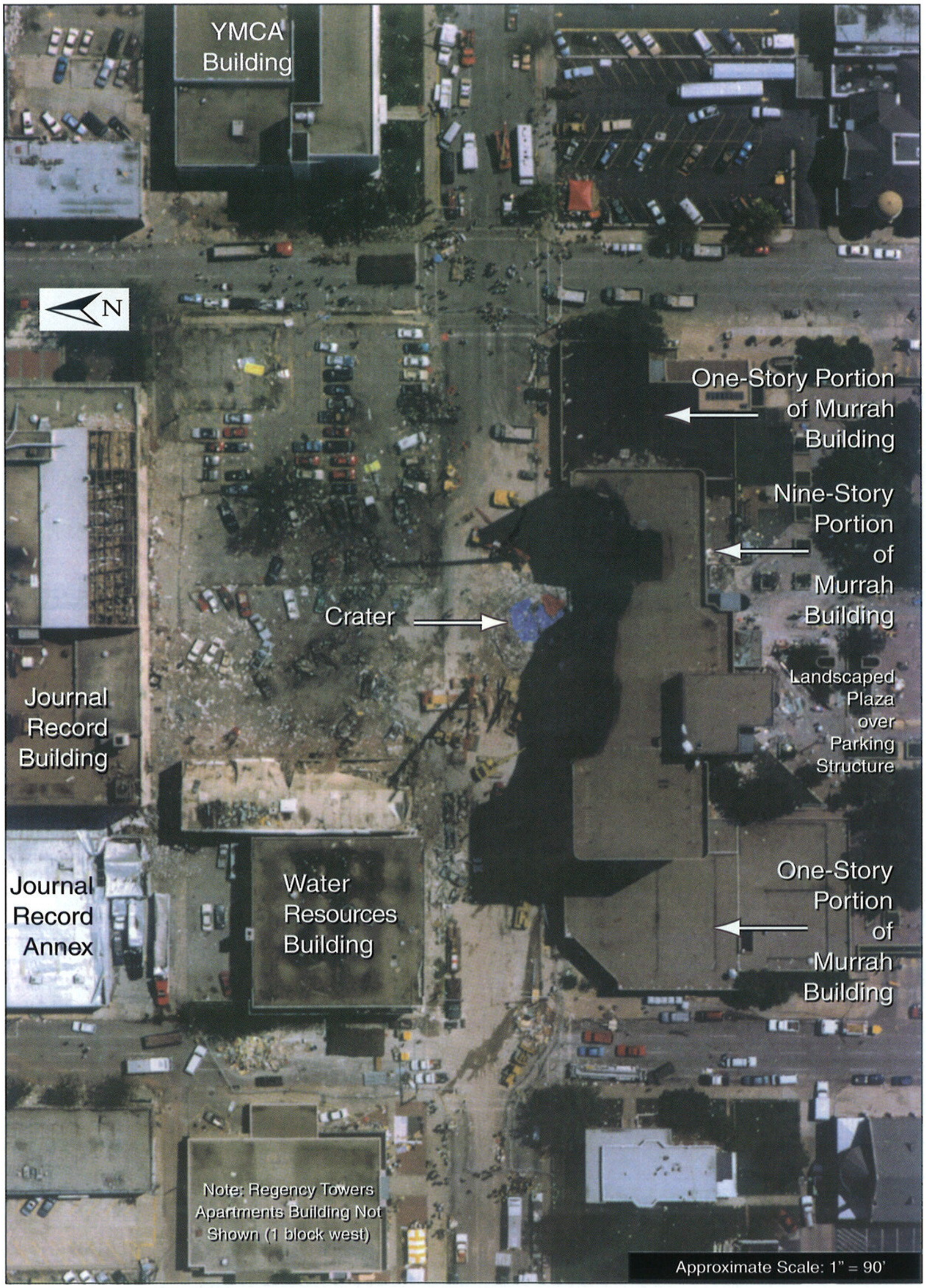


Figure 1-13 Aerial view of damaged buildings.

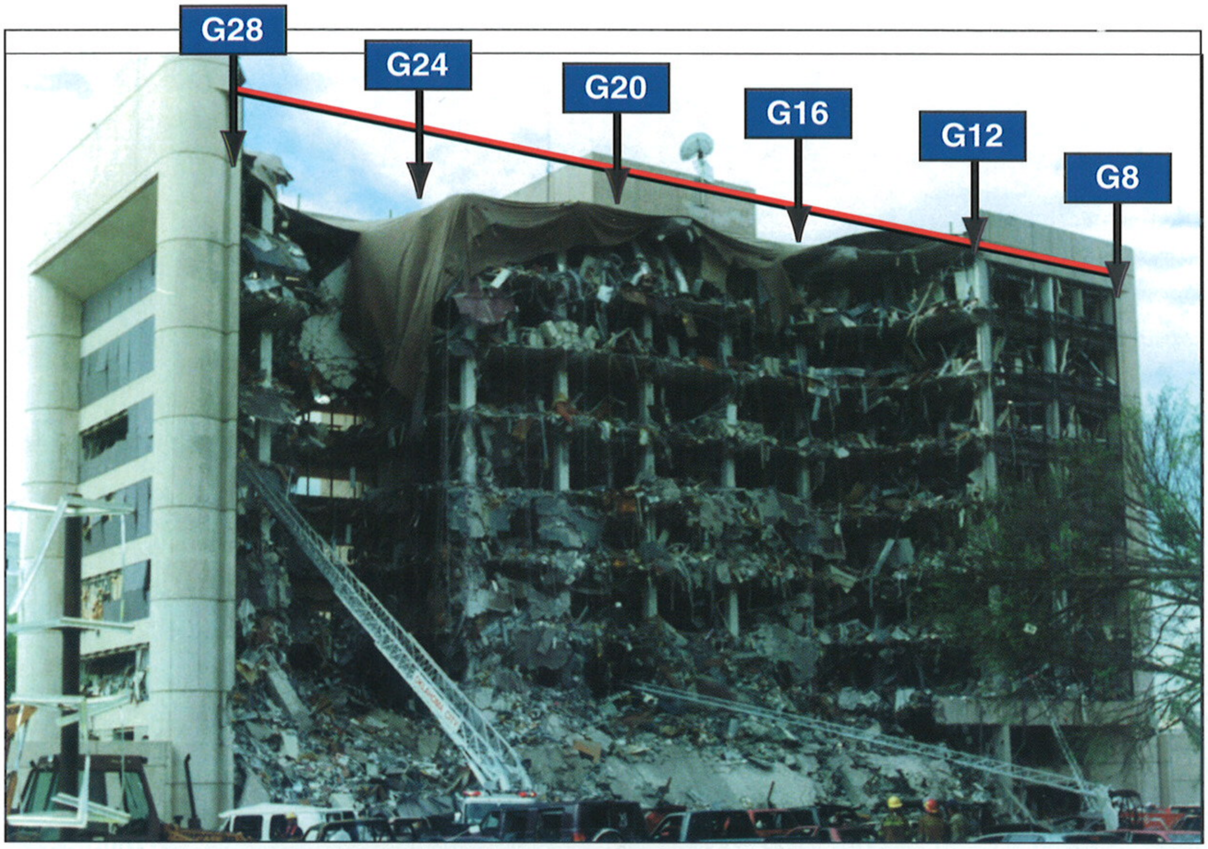


Figure 1-14 *Damage to north and east sides of Murrah Building.*

Figure 1-15 *Damage to southeast portion of Murrah Building.*

Figure 1-16 Damage to southwest portion of Murrah Building.

Figure 1-17 Damage to Water Resources building, looking east.

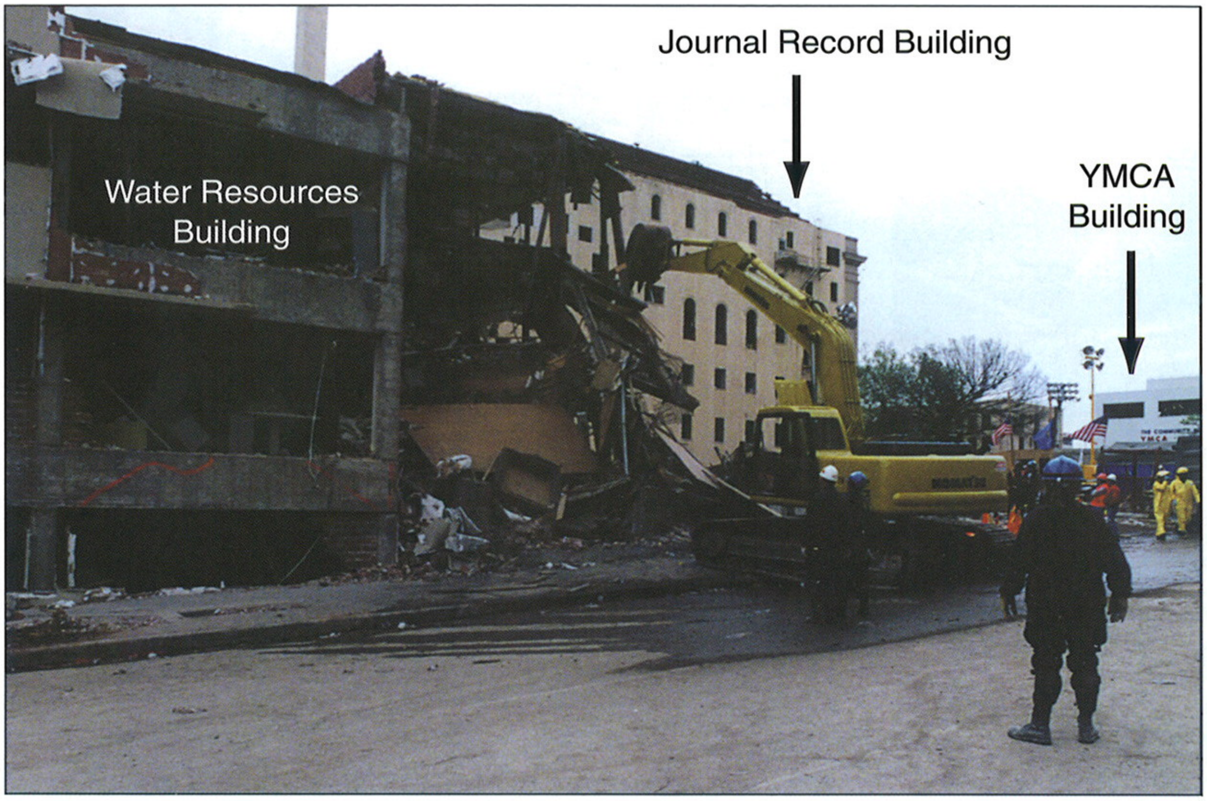


Figure 1-18 *Damaged Water Resources building, Journal Record building, and YMCA building, looking northeast.*

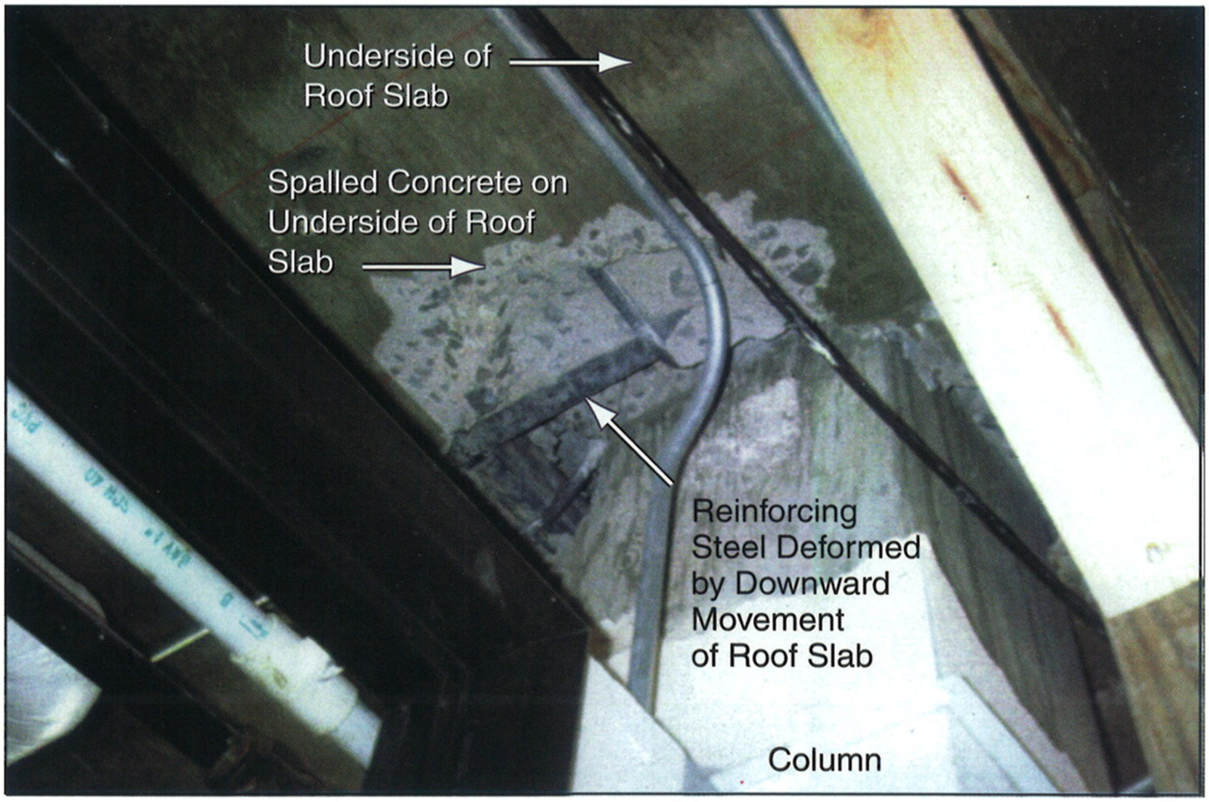


Figure 1-19 *Damage to underside of roof slab in Water Resources building.*

Journal Record Building

The Journal Record building, located due north of the nine-story portion of the Murrah Building, sustained significant damage to its exterior skin. The steeply pitched concrete roof panels facing the blast, most windows, and small portions of the masonry were destroyed. As in the Water Resources building, windows were blown inward and exterior masonry sections failed outward. In a reaction similar to that of the exterior skin of the Water Resources building, the concrete roof panels disengaged from the purlins in a "rebound" motion due to the low-pressure portion of the blast wave in combination with the rebound of the purlin/truss support system (see Figure 1-20). Localized rebound of each panel spanning the purlins, combined with other actions, may have contributed to disengagement of the roof panels. Steel roof purlins, spanning between steel roof trusses, exhibited visible evidence of yielding in the form of large permanent distortions.

A one-story light-metal steel building just west of the main Journal Record building housed the presses. This building, referred to as the Journal Record Annex (see Figure 1-13), was badly damaged, sustaining severe deformations of roof and wall members (see Figure 1-21). Also, an unreinforced masonry wall on the north side of the Annex was damaged. This wall was quickly demolished and removed to reduce the hazard to workers. All of the damage to the Annex was attributable to blast effects.

YMCA Building

The YMCA building (see Figure 1-22) sustained less damage than the Water Resources building or the Journal Record building. However, with few exceptions, the glazing of the YMCA building was blown inward and destroyed .

Structural damage to a light-metal roof system that had been added was visible at a few locations. This damage was due primarily to the downward force on the roof, which caused localized buckling of the top chords of roof joists (see Figure 1-23).

Regency Towers Apartments Building

In the Regency Towers Apartments building, located about one block from the blast, further from the blast than the aforementioned buildings, all of the windows were destroyed (see Figure 1-24). There was no structural damage to this 1960's vintage reinforced concrete building. However, damage in the building at the air supply/exhaust system (see Figure 1-25) showed that the blast effects propagated through the mechanical ducting system and into the building. Also, some gypsum wallboard and stud partition walls were blown over inside the building.

1.4.5 INTERIOR

Observed failure boundaries for the floor slabs and roof slab are shown in Figure 1-26. All floor and roof panels between Column Lines 12 and 28 and Column Lines F and G failed, either partially or completely. In addition, with the loss of Column F24, all floor and roof panels between Column Lines 22 and 26 and Column Lines E and F also failed. Finally, floor panels on the second and third floors failed between Column Lines 18 and 22 and Column Lines E and F. In summary, out of a total of 180 floor and roof panels at the second floor and above, 112 failed either partially or completely.

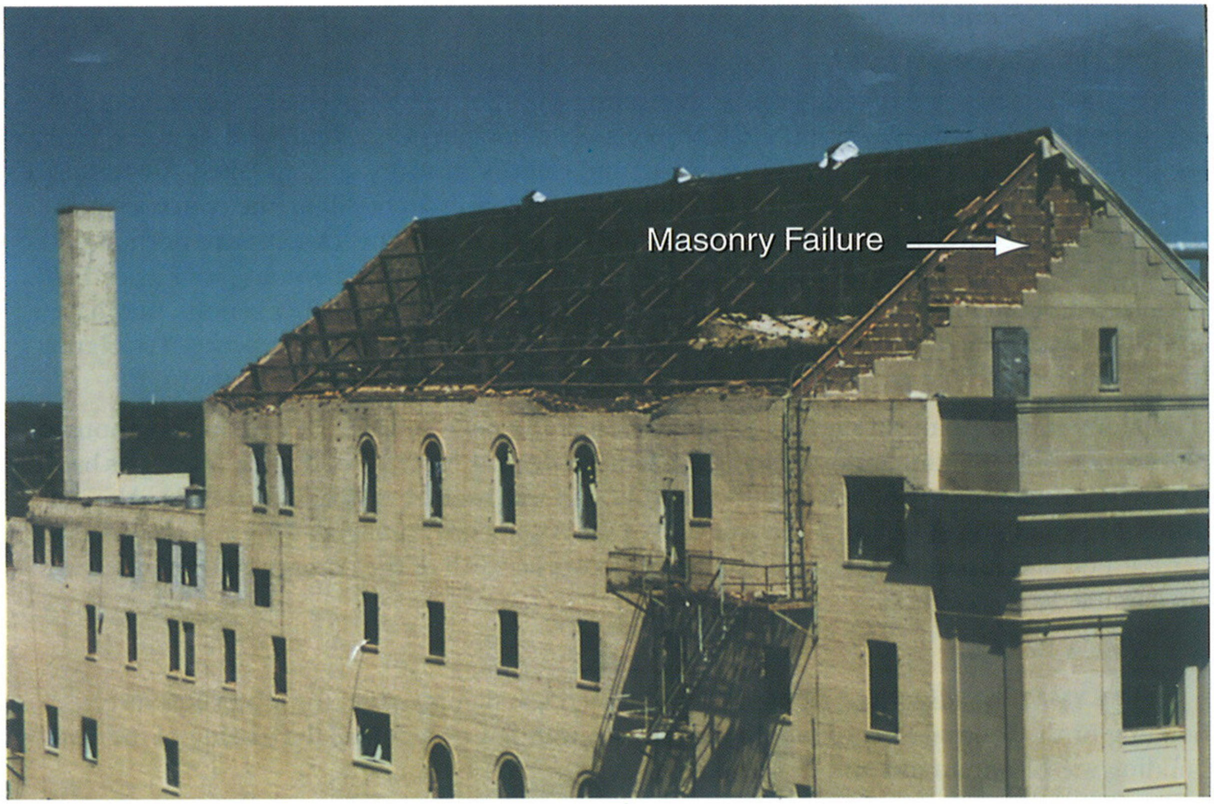


Figure 1-20 *Journal Record building – damage to roof on south side and masonry failure on east side.*

Figure 1-21 *Damage to roof and walls of one-story light-metal steel building (Journal Record Annex) on west side of main Journal Record building.*

Figure 1-22 South face of YMCA building.

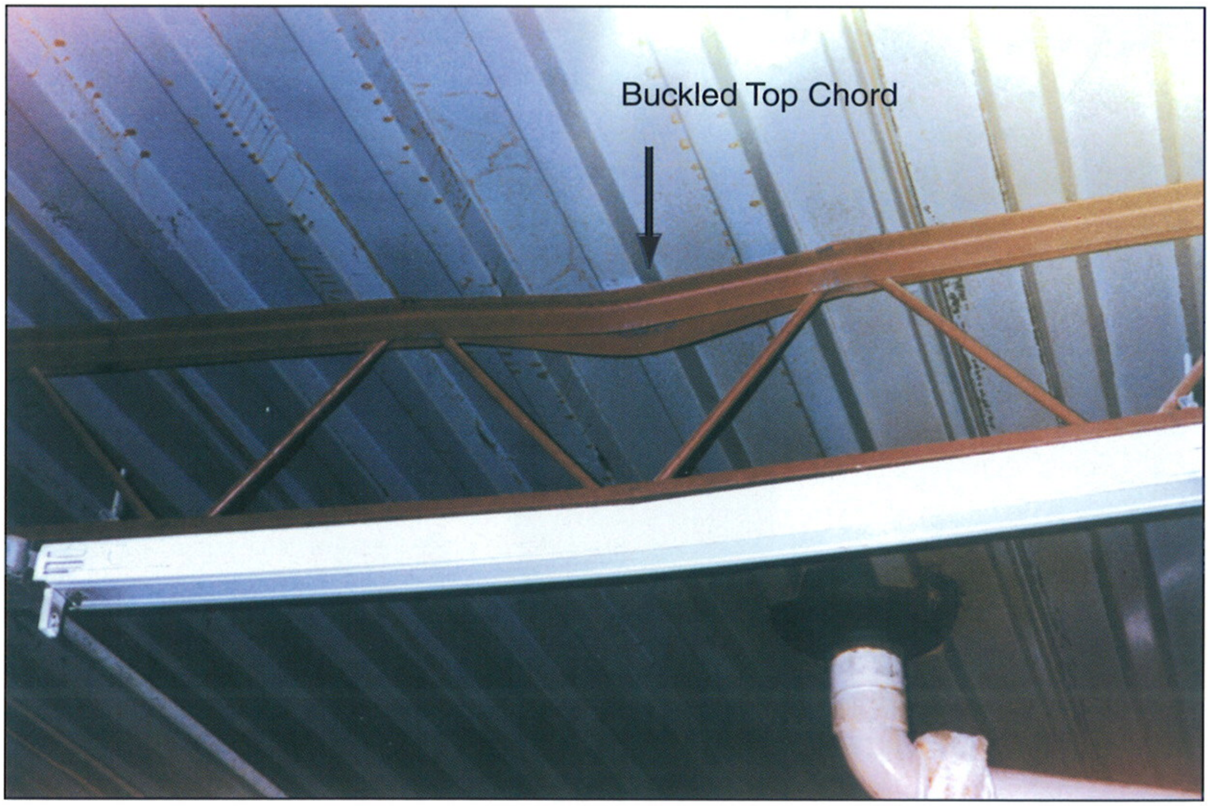


Figure 1-23 Bar joist in upper story of YMCA building.

Figure 1-24 *Regency Towers Apartments building.*

Figure 1-25 *Damage to utility shaft of Regency Towers Apartments building.*

1.4 General Scope of Damage to Nine-Story Portion of Murrah Building

This section describes the spatial distribution of damage to the nine-story portion of the Murrah Building. Damage resulted from either direct blast effects or subsequent progressive collapse.

1.4.1 Global Response

Lateral and torsional stiffnesses of the building were concentrated in the core section along the south wall (Column Line E) and to a lesser extent in the air shafts at the four corners of the nine-story portion. There is no evidence to suggest that any significant lateral or torsional displacements of a global nature occurred during the collapse.

1.4.2 End Walls

Structural details and architectural treatment of the end walls (east and west faces) are described in Section 1.2. By far, the largest amount of damage occurred at the east end wall. Some granite panels of the infill wall were dislodged at the third through the sixth floors. All granite panels were totally dislodged at the third floor, and approximately half the panels at the seventh floor were fractured. Most panels that remained in place randomly failed in flexure, either inward or outward in no particular order. All glazing panels at the second floor failed. All of the granite panels on the west end wall remained in place, but many failed in flexure, either inward or outward.

Structural damage to the end walls was limited to the interior face of the east wall at Column Line G. Typically, damage occurred at each floor level where the longitudinal reinforcement in the spandrel beams and in the transfer girder pulled away from the west side of the northeast air shaft at Column G28. In addition, the east wall suffered direct blast damage at the second floor. There is no photographic evidence to suggest that either end wall underwent significant displacement during the blast or subsequent collapse. However, it has been reported that measurements taken during debris removal showed the northeast air shaft to be approximately 2 inches out of plumb.

1.4.3 South Face

Damage to the south face of the office building was limited to failure of glazing, mullions, and door frames. The fact that much of the south face consists of the core structure of the building and contains concrete masonry partition walls undoubtedly played a significant role in limiting damage to this face.

1.4.4 North Face

Damage to the north face (Column Line G) resulted from the direct effects of blast and from the subsequent progressive collapse. Total collapse of the north face occurred from Column Line 12 to Column Line 28. The only exception was a segment of the transfer girder (third floor) which remained in place between Column Lines 12 and 14 (see Figure 1-26).

Between Column Lines 8 and 12, the columns, spandrel beams, and transfer girder were relatively undamaged. Although all glazing on the north face failed, the mullions at and above the third floor between Column Lines 8 and 12 remained in place.

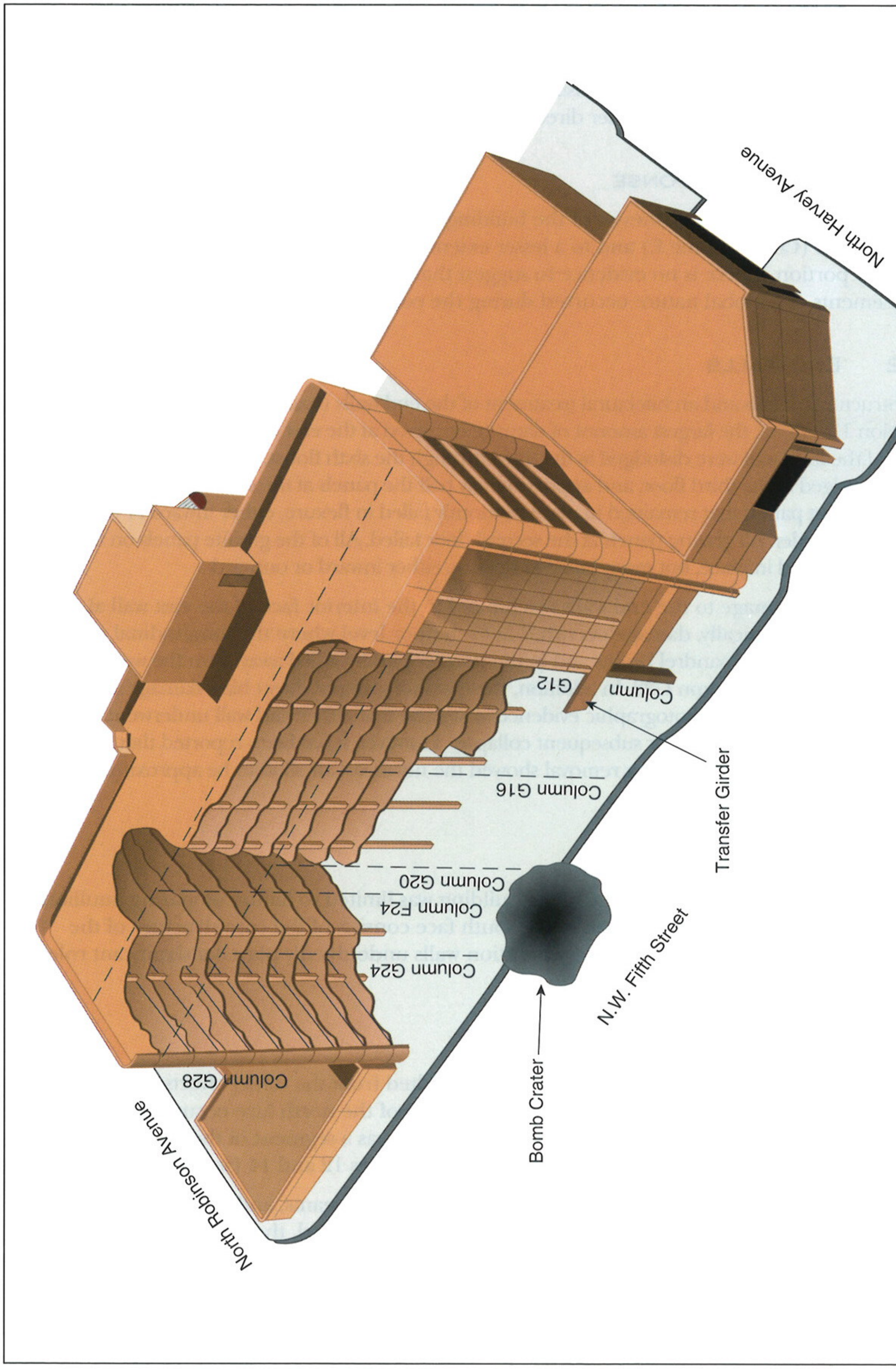


Figure 1-26 Failure boundaries of roof/floor slabs in Murrah Building.

1.4.5 INTERIOR

Observed failure boundaries for the floor slabs and roof slab are shown in Figure 1-26. All floor and roof panels between Column Lines 12 and 28 and Column Lines F and G failed, either partially or completely. In addition, with the loss of Column F24, all floor and roof panels between Column Lines 22 and 26 and Column Lines E and F also failed. Finally, floor panels on the second and third floors failed between Column Lines 18 and 22 and Column Lines E and F. In summary, out of a total of 180 floor and roof panels at the second floor and above, 112 failed either partially or completely.

All suspended ceiling panels and much of the ductwork located above these panels were removed by blast effects. In the core area, concrete masonry partition walls were heavily damaged, particularly on the first three floors. Figure 1-27 shows damage to one of the elevator shafts in the core area near Column Line 16.

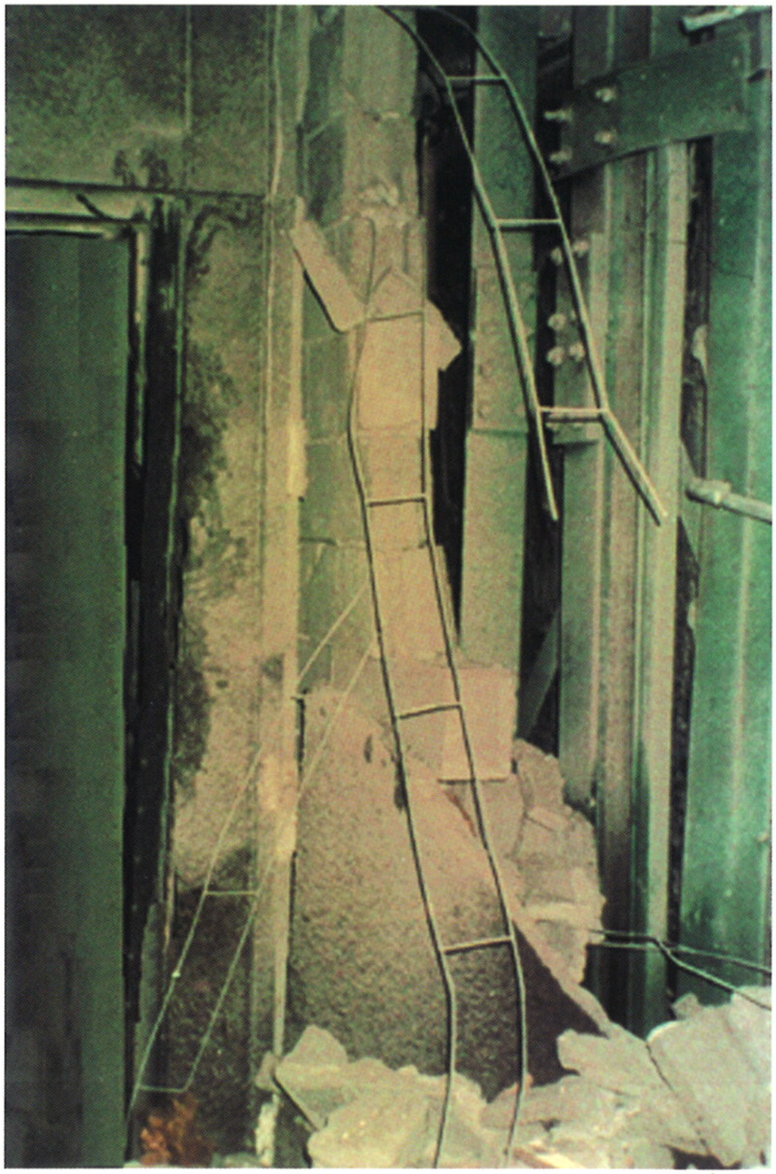


Figure 1-27 Damage to masonry wall at elevator shaft in Murrah Building.

2 Data Collection

2.1 Structural Drawings and System

Structural drawings show that the Murrah Building consisted of cast-in-place ordinary reinforced concrete framing with conventionally reinforced columns, girders, beams, slab bands, and a one-way slab system. Exterior spandrels supporting the exterior curtainwall were exposed concrete with a vertical-board-formed finish. The lateral load resisting system for wind forces was composed of reinforced concrete shear walls located within the stair and elevator systems on the south side of the building. Although the design was not required by the governing building code or by the owner to consider blast loading or earthquake loading, the required wind-load resistance provided substantial resistance to lateral load.

According to general notes on the structural drawings, the Murrah Building project was all reinforced concrete that was proportioned, fabricated, and delivered in accordance with the American Concrete Institute (ACI) *Building Code Requirements for Reinforced Concrete* (ACI 318-71, Reference 2). The yield strength for ties, # 3 bars, and concrete reinforcing for stirrups was 40,000 pounds per square inch. The yield strength for all other deformed bars and all welded wire fabric was 60,000 pounds per square inch. The 28-day concrete compressive strengths included 3,000 pounds per square inch for foundation and equipment bases and 4,000 pounds per square inch for structural beams, slabs, columns, walls, counterforts, pilasters, spread footings, and parking garage exterior walls. General notes also required all reinforcing bar splices to be lapped 30 bar diameters unless otherwise noted.

As indicated on the plans, the design live loads for the project are as follows:

Project Element	Load (pounds per square foot)
Roof	20
Office Areas (exclusive of 20-pounds-per-square-foot load provision for movable partition)	50
Parking Structure Floors	50
Corridors, Stairs, and Lobbies	100
Plaza Pedestrian Walks	100
Mechanical Equipment Spaces, Elevator Machine Rooms, Equipment Load	150
Vehicle Maintenance	150
Street-Level Sidewalk and Approaches	250

(As noted previously, blast and earthquake loads were not prescribed by the building code and thus were not considered.)

Design wind loads for the projected building areas are as follows:

Height (feet)	Load (pounds per square foot)
less than 30	25
30 - 50	30
50 - 100	40
above 100	45

Because the blast occurred on the north face of the nine-story portion of the Murrah Building, this report describes the configuration of the structural system of the north face, along Column Line G, as well as the intermediate columns along Column Line F, primarily in the area between Column Lines 16 and 28.

The 117 architectural drawings and 40 structural drawings for the project are carefully detailed, well prepared, and well coordinated. The level of structural detailing and the use of schedules with full dimensions for all slab, T-beam, spandrel beam, transfer girder, and column reinforcing bars are significantly better than normally expected for buildings of this type. Although a complete structural design check was not undertaken, those components that were reviewed were designed in accordance with accepted standards and ACI 318-71.

Selected details concerning the floor framing, roof structure, interior columns, foundation, and dowels are provided in Appendix B. Also provided in Appendix B are the results of a dead load analysis the BPAT performed using information shown on the structural drawings.

2.2 Concrete and Reinforcing Bar Samples

On May 12, 1995, the BPAT visited the Oklahoma County Sheriff's Firing Range to interview personnel and view debris from the Murrah Building. During this visit, photographs taken soon after the explosion by several law enforcement organizations were reviewed. Also, several pieces of building debris were inspected.

Inspection of remaining pieces of concrete debris disclosed that there were a few "chunk" samples and sections of spandrel beams near the parking lot of the Firing Range. Also, a few pieces of deformed reinforcing bar were stacked near the concrete debris. Sheriff's personnel confirmed that the concrete and the reinforcing bars had come from the Murrah Building.

Other pieces of debris, including some large slab sections, were stored on one of the firing ranges north and west of the main parking lot. Sheriff's personnel confirmed that these pieces of concrete also had been taken from the Murrah Building.

Locations were marked on six concrete debris samples where cores were to be taken. In addition, a seventh sample, a chunk of concrete, was marked where a core was to be taken. Several sections of reinforcing bars were marked to be taken as samples.

On May 12, 1995, Terracon Consultants, Inc., of Oklahoma City took six 6-inch cores from the marked areas. After the cores had been taken in the field, they were packed in a plastic cooler and shipped to Construction Technology Laboratories, Inc. (CTL), in Skokie, Illinois, where selected samples were tested. Similarly, reinforcing bar samples were put in a heavy plastic shipping tube and sent to CTL for testing. All samples arrived at CTL on May 15, 1995. The necessary chain-of-custody documentation was maintained for all samples.

2.3 Photographs and Video Tapes

In addition to conducting a visual inspection, the BPAT reviewed photographs and video tapes recorded on April 19, 1995, and during the following rescue and recovery period. These records were useful in establishing the performance of the building.

2.4 Audio Tape

At 9 a.m. on April 19, 1995, a hearing on water rights was convened in the Water Resources building, which is located at the northeast corner of North Harvey Avenue and N.W. Fifth Street, less than block from the site of the explosion (see Figure 1-3). An audio tape recorder, used to document the hearing, was started approximately 2 minutes before the blast occurred.

The audio recorder captured the sound of the blast. An analysis of the recording is shown in Figure 2-1. As can be seen in this figure, the blast was followed by a brief period of moderate noise, then a period of about 3 seconds during which the noise level exceeds the range of the recorder. This 3-second period is interpreted to be the time it took for the building to collapse.

2.5 Interviews

In a comprehensive effort to obtain information, members of the BPAT interviewed several key individuals and groups of people. These interviews were conducted in an effort to determine the original design parameters for the building and to obtain information about events as the building responded to the blast loading and information about the damage present in the building immediately after the collapse.

2.5.1 Structural Engineer of Record

Dr. W. Gene Corley (ASCE) interviewed the Structural Engineer of Record for the Murrah Building project, Paul Kirkpatrick, P.E., on May 11, 1995. Mr. Kirkpatrick noted that the original design parameters did not require any consideration of blast resistance, earthquake, or other extreme loading not required by building codes at the time. Rather, the structure was to be designed for normal office building loading in Oklahoma City.

Mr. Kirkpatrick stated that soon after the blast, he was called to the site to assist with stabilizing the debris. He noted that the primary failure involved collapse of the north portion of the office structure. Two panels in the south half of the building also collapsed. Mr. Kirkpatrick did not observe any damage that suggested significant lateral movement of the structure. He saw no visible signs of hinging at the tops or bottoms of any remaining columns. Also, there were no visible indications that shearwalls had hinged significantly.

In general, Mr. Kirkpatrick believed that the collapse was initiated by failure of the columns along the north side of the building. In his opinion, two or more columns may have been destroyed in the vicinity of where the blast originated. He felt that perhaps a third column (supporting a transfer girder), as well as one interior column, could have been destroyed by the blast itself.

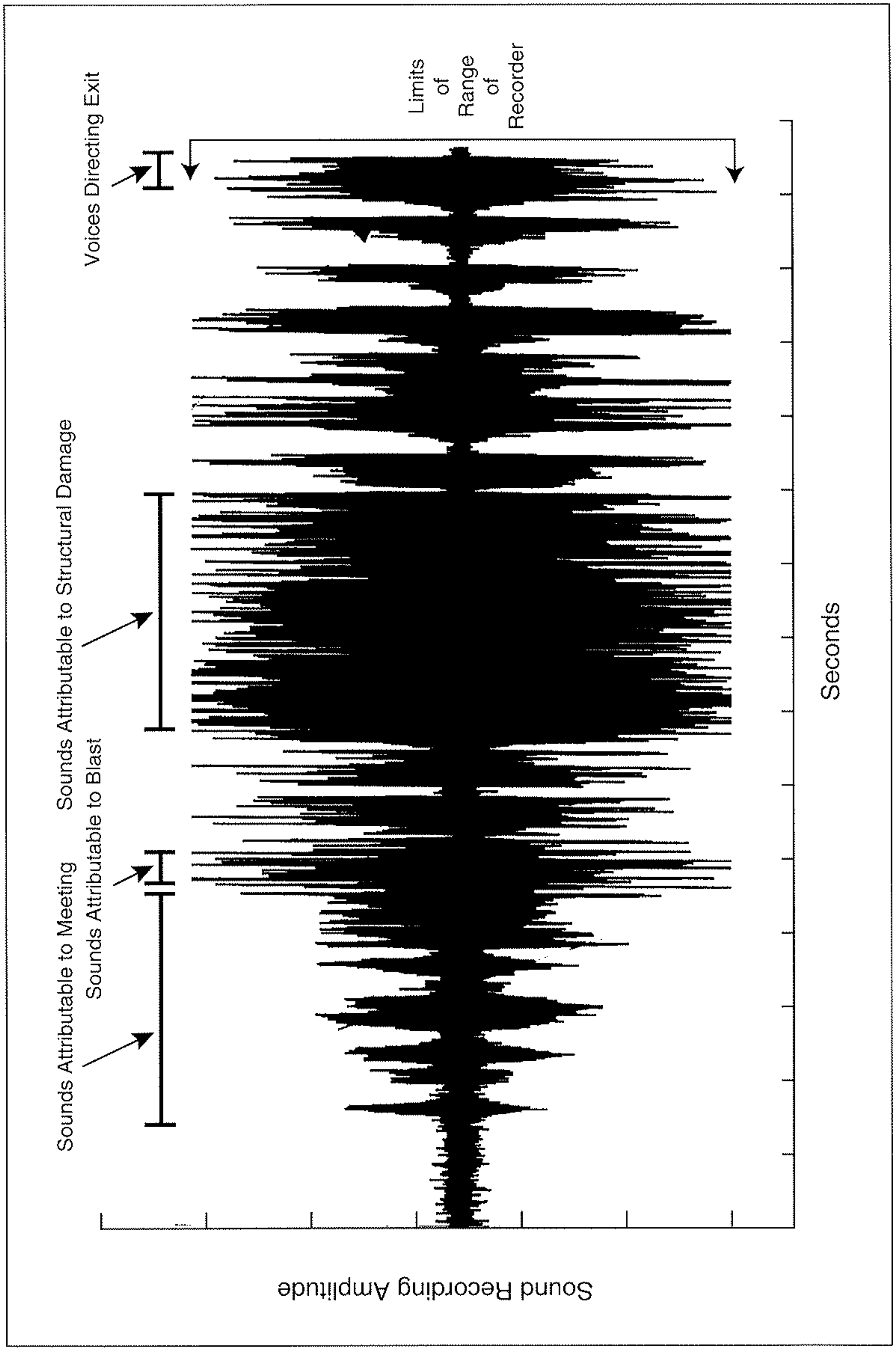


Figure 2-1 *Analysis of tape recording from Water Resources building.*

2.5.2 Architect of Record

The Architect of Record was William M. Shaw, formerly of Oklahoma City. Attempts were made to contact Mr. Shaw at his current office in Elk City, Oklahoma; however, he was unavailable to participate in an interview.

2.5.3 Water Resources Board Employee

On May 11, 1995, an interview was held with Ken Morris, an employee of the Oklahoma State Water Resources Board. Mr. Morris was in the Water Resources building at the time of the blast. He stated that when the explosion occurred, he noticed the building shaking, fluorescent lights and windows exploding, ceiling tile falling, and walls blowing down. He immediately smelled gas and nitrates.

Although Mr. Morris was knocked to the floor and was aware that there was substantial damage to his surroundings, he never did "hear" the explosion. After helping other employees leave, he left the Water Resources building and walked about one block northwest, to where he had parked his truck. The truck had sustained severe damage, including dented doors and broken windows.

2.5.4 GSA Representatives

On May 13, 1995, the BPAT interviewed Robert Hill of Brochette, Davis, and Drake, an engineering consultant to GSA, and John McRoberts, a GSA employee. They noted that there was nothing unexpected about failure surfaces in the structure. For the most part, the hinging surfaces indicated a flexural failure accompanied by ripping out of the bottom reinforcement as the collapse progressed. They noted that the concrete appeared to be normal-weight aggregate.

2.5.5 Robert Cornforth, P.E.

Robert Cornforth, an Oklahoma City structural engineer, provided very helpful insight concerning damage to surrounding buildings. Mr. Cornforth was called in to inspect a number of buildings damaged by the explosion. These included the 24-story high-rise Regency Towers Apartments building, which has a reinforced concrete frame; the Journal Record building, which also has a reinforced concrete frame; the one-story Journal Record Annex, which has a light-metal steel frame; and the YMCA building, which has a reinforced concrete frame and is across the street from and northeast of the Murrah Building site. In addition, Mr. Cornforth had inspected a number of low-rise, unreinforced masonry buildings and steel-frame buildings in the area.

Mr. Cornforth noted that although there was nonstructural damage to many buildings, he found little significant structural damage to engineered structures more than one block away from the Murrah Building. Specifically, the 24-story Regency Towers Apartments building, the main multi-story portion of the Journal Record building, and the YMCA building had no damage to their structural frames. Several non-load-bearing walls inside those three buildings and portions of the roofs were damaged or destroyed, but the structural frames were intact and essentially undamaged. None of these buildings was in danger of general collapse.

It was noted that the one-story light-metal steel building referred to as the Journal Record Annex was badly damaged. There was severe damage to exterior walls and the roof, and the loading dock enclosure was near collapse.

2.5.6 GSA Structural Consultant

The firm of Brockett, Associates of Dallas, Texas, serves as Structural Consultant to GSA, the owners of the building. Robert Hill, Structural Engineer, provided information to the BPAT.

Mr. Hill noted that failure surfaces he observed did not indicate shear strength controlled the failure of the slabs and beams. Most surfaces were at a slope that is generally associated with flexural behavior. He noted that he had not been able to observe the failure patterns for Columns G-16, G-20, and G-24 in the lower two stories.

When asked about the conditions of columns still standing, Mr. Hill stated that there was no indication the columns had hinged at the tops and bottoms, as would be the case if the building developed an overall hinging mechanism because of lateral load. Rather, he believed that the general mechanisms that developed were associated with loss of one or more columns from blast loading.

2.5.7 FEMA Engineering Consultant

Engineer David Hammond of Palo Alto, California, an engineering consultant working for FEMA, provided the team with his observations. Mr. Hammond also confirmed there was no evidence of flexural hinging at the tops and bottoms of columns. Consequently, he did not believe the building developed a mechanism from lateral load.

Based on his observations, Mr. Hammond offered the opinion that at least one column near the blast had been destroyed by brisance (the shattering effect of the blast). He also believed that at least one and perhaps two other columns had been knocked out by the blast.

2.6 Tests of Materials

2.6.1 Concrete

Five of the cores taken from the concrete from the Murrah Building were selected for compression testing. In addition, two cores were taken from the chunk sample; one of these was tested in compression and the other was subjected to petrographic examination. Results of these tests are shown in Table 2-1.

Although there was evidence of damage from blast and/or from handling stresses, visual inspection of the concrete indicated that it met or exceeded the quality called for in the design specifications. Subsequent compression test results indicate that the concrete strength was well in excess of the 4,000 pounds per square inch called for in the design specifications.

As noted in the report provided in Appendix C, the petrographic evaluation indicated that the concrete contained normal-weight aggregate and was of the quality required in the design specifications.

2.6.2 Reinforcing Bars

Several pieces of reinforcing bars were recovered from the debris of the building. A few lengths of straight bar were tested in tension. Results of these tests are presented in Table 2-2. In all cases, the yield stresses and strengths measured for the bars were greater than the minimums specified and easily met the requirements of the design specifications.

	Core Identification		
	No. 3	No. 5	No. 7
Nominal Maximum Aggregate Size (inches)	3/4	3/4	3/4
Concrete Age at Test (approximate days)	Not Stated	Not Stated	Not Stated
Moisture Condition at Test	Dry	Moist	Moist
Orientation of Core Axis in Structure	Not Stated	Not Stated	Not Stated
Diameter 1 (inches)	5.70	5.71	3.26
Diameter 2 (inches)	5.68	5.71	3.25
Average Diameter (inches)	5.69	5.71	3.25
Cross-Sectional Area (square inches)	25.42	25.63	8.30
Length Trimmed (inches)	10.3	7.6	6.4
Length Capped (inches)	10.6	7.9	6.6
Weight in Air (pounds)	22.2	16.7	4.5
Weight in Water (pounds)	12.7	9.7	2.6
Density (pounds per cubic foot)	146.4	149.5	147.3
Loading Rate (pounds per square inch per second)	35	35	35
Maximum Load (pounds)	107,500	177,500	46,000
Uncorrected Compressive Strength (pounds per square inch)	4,230	6,930	5,540
Ratio of Capped Length to Diameter (L/D)	1.86	1.38	2.03
Correction Factor (ASTM C42)	0.989	0.945	1.000
Compressive Strength Corrected for L/D (pounds per square inch)	**4,180**	**6,550**	**5,540**
Fracture Pattern	Shear	Conical	Columnar
Compressive Strength Corrected for L/D and for Cylinder vs. Core Ratio (pounds per square inch / 0.85)	**4,920**	**7,710**	**6,520**

Note: Core No.1 had a vertical crack before testing. Core No. 5 had a 4 1/2-inch piece of #4 reinforcement bar with 1 1/2-inch cover running transversely 4 inches from the top surface of the core.

Table 2.1 Test results for concrete used in Murrah Building.

	Specimen B	Specimen H
Size	#8	#5
Design Yield Stress (thousands of pounds per square inch)	60	60
Measured Yield Stress (thousands of pounds per square inch)	71.8	65.2
Yield Load (thousands of pounds)	56.7	20.2
Ultimate Load (thousands of pounds)	84.9	33.3
Ultimate Stress (thousands of pounds per square inch)	107	107
Elongation (percent)	9.8	11.1
Gage Length	7-7/8	7-3/4
Modulus of Elasticity (thousands of pounds per square inch)	28,500	28,500

Table 2.2 Test results for flexural reinforcement used in Murrah Building.

2.6.3 Geotechnical Material Near the Crater

Architectural plans provide soil test boring data obtained along N.W. Fifth Street. The borings were taken approximately 19.5 feet south of the street centerline. Crater survey measurements located the center of the crater to be approximately 28 feet south of the street centerline. Also, the crater was located approximately midway between the locations of Boring No. 2 and Boring No. 3 on the architectural plans. According to these two borings, the average thickness of the combined asphalt/concrete pavement is approximately 18 inches.

During the crater survey, pavement thickness was measured to include approximately 11 inches of asphalt over 7 inches of concrete at the north tip of the crater. Below the concrete layer, Boring No. 2 indicates that the soil is a brown-red silty sand to a depth of 2.2 feet and a red clay with sandstone down to 14 feet. Boring No. 3 shows a loose fine red sand to a depth of 6 feet and a loose fine to medium grain tan sand to a depth of 8.3 feet.

3 *Analysis*

3.1 Method of Review

The analytic assessment of the Murrah Building bombing proceeded in two steps. First, weapons effects information and structural dynamic methods were used to calculate the blast loading and response of critical elements, as discussed in Section 3.2. The progressive collapse of the building was then examined by static frame analyses in the absence of damaged or destroyed elements to determine possible failure mechanisms, as discussed in Section 3.3. The findings are summarized in Section 3.4.

3.2 Blast Loading and Response

3.2.1 Blast Effects

The explosive device was contained in an enclosed truck parked on the paved street along the north side of the Murrah Building (N. W. Fifth Street). The blast caused the collapse of part of the nine-story portion of the Murrah Building as well as damage to a number of nearby structures. In this section, the effects of the blast are quantified for engineering analysis. For the purposes of this report, the quantification is performed with methods used for the analysis of conventional weapons effects on structures (TM5-855-1, Reference 28).

The calculation of blast loading begins with the estimation of the yield or quantity of explosives detonated. For bursts near the ground surface, the yield or quantity is usually inferred from the dimensions of the crater formed (see Figure 3-1). In this report, the engineering survey of the crater forms the basis of this inference.

The crater was approximately 28 feet in diameter and 6.8 feet in depth, as shown in Figure 3-2. The center is about 7 feet east and 14 feet north of Column G20, as shown in Figure 3-3. According to the design drawings and observations on site, the thickness of the pavement was 18 inches and the underlying soil was dry sandy clay. From information about the truck reported to have contained the explosive device, the center of the explosive is estimated to have been 4.5 feet above the ground surface, as shown in Figure 3-2.

As shown in Table 3-1, the detonation of a spherical charge of trinitrotoluene (TNT, the standard by which the energy of various explosives is measured) weighing approximately 4,000 pounds at 4.5 feet above 18-inch-thick pavement on soil results in a crater whose dimensions are consistent with those measured at the Murrah Building site. In Table 3-1, the crater dimensions for pavement on soil are an average of those for massive concrete and for the dry sandy clay alone, weighted in proportion to the depths of the two materials in the crater. This weighting is substantiated by the results of ongoing research concerning craters in pavements based on soil.

Figure 3-4 presents an indication of the blast loading in the general neighborhood of the Murrah Building. It specifically shows the incident free-field overpressure contours corresponding to the surface detonation of 4,000 pounds of TNT at the location of the bomb crater. These contours correlate approximately with the level of damage shown for buildings in the neighborhood. However, this damage is also a function of load modification by nearby buildings and the resistance of the buildings themselves.

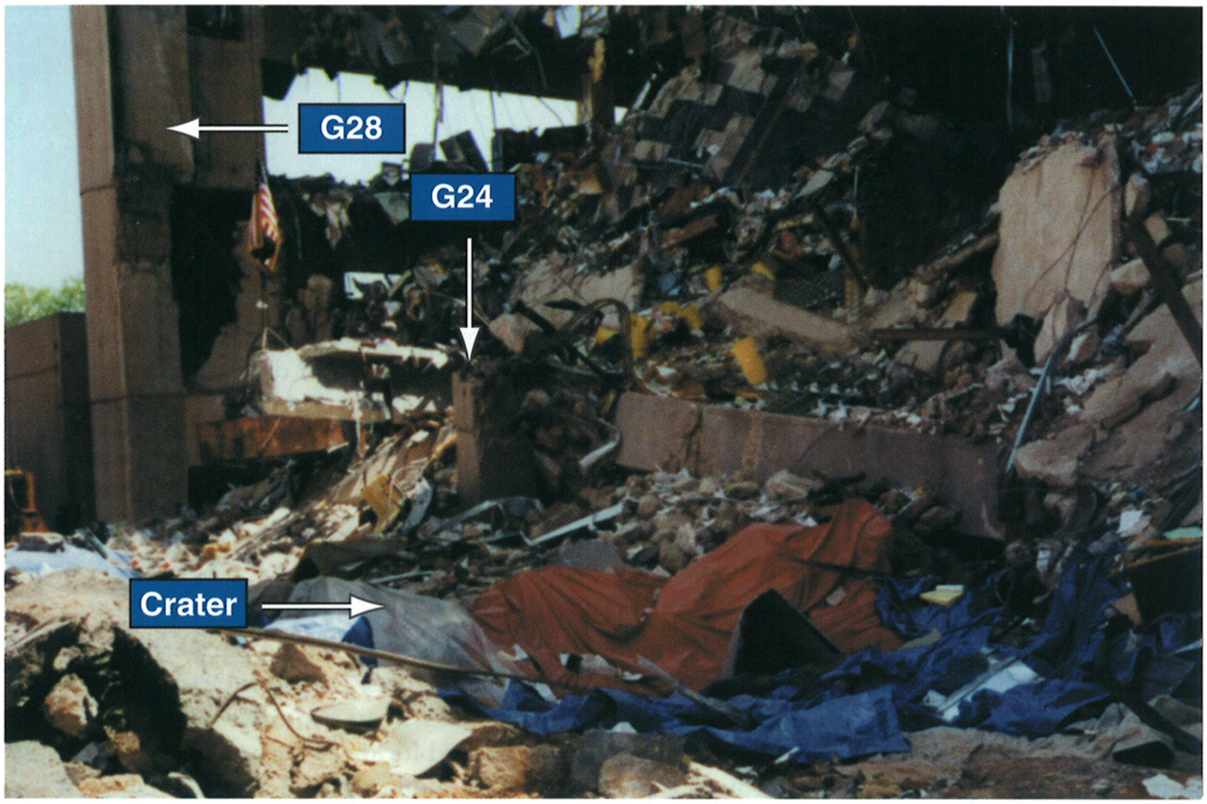


Figure 3-1 Bomb crater (covered by tarp).

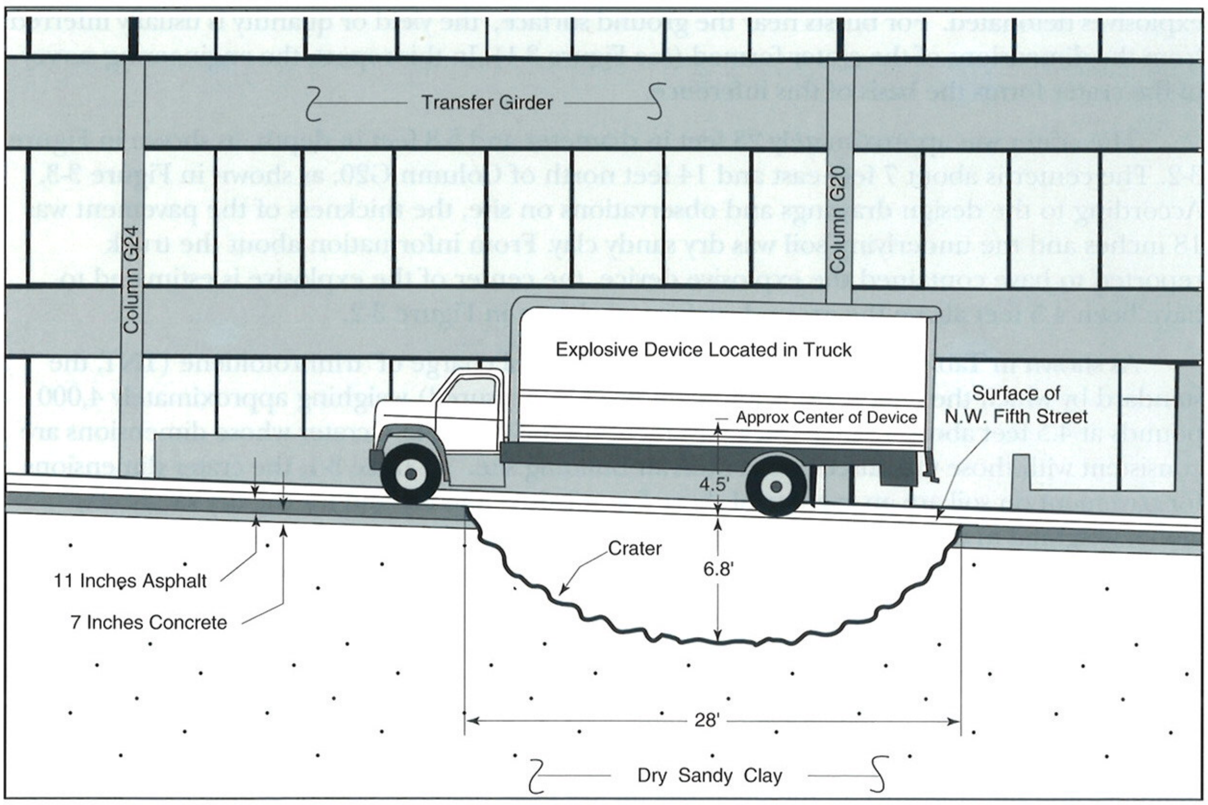


Figure 3-2 Approximate dimensions of crater at north face of Murrah Building.

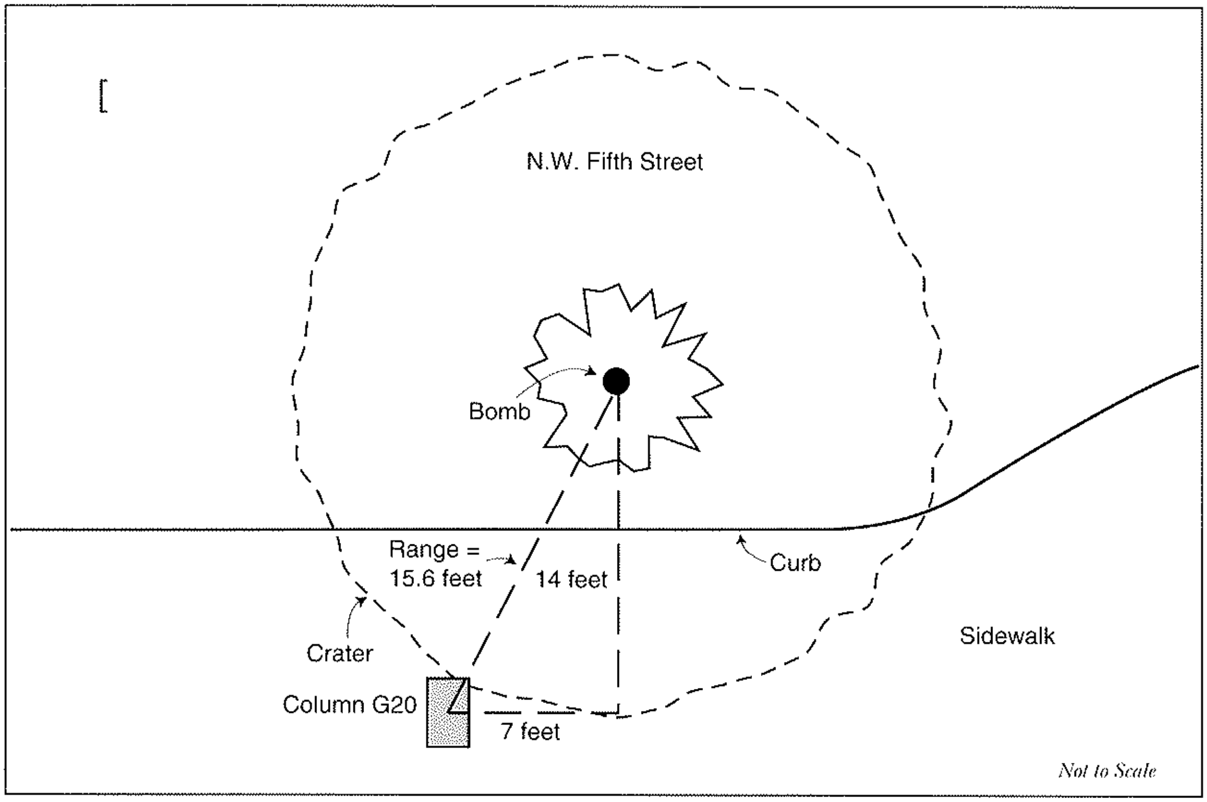


Figure 3-3 *Proximity of Column G20 to location of bomb (plan view).*

Condition	Depth (feet)	Diameter (feet)
4,000 Pounds of TNT on Massive Concrete	2.6	13
4,000 Pounds of TNT on Dry Sandy Clay	8.5	31
4,000 Pounds of TNT — 18 Inches of Pavement on Soil	7.2	27
Measured at Murrah Building	6.8	28

Table 3-1 *Estimate of yeild from crater dimensions (TM5-855-1, Reference 28).*

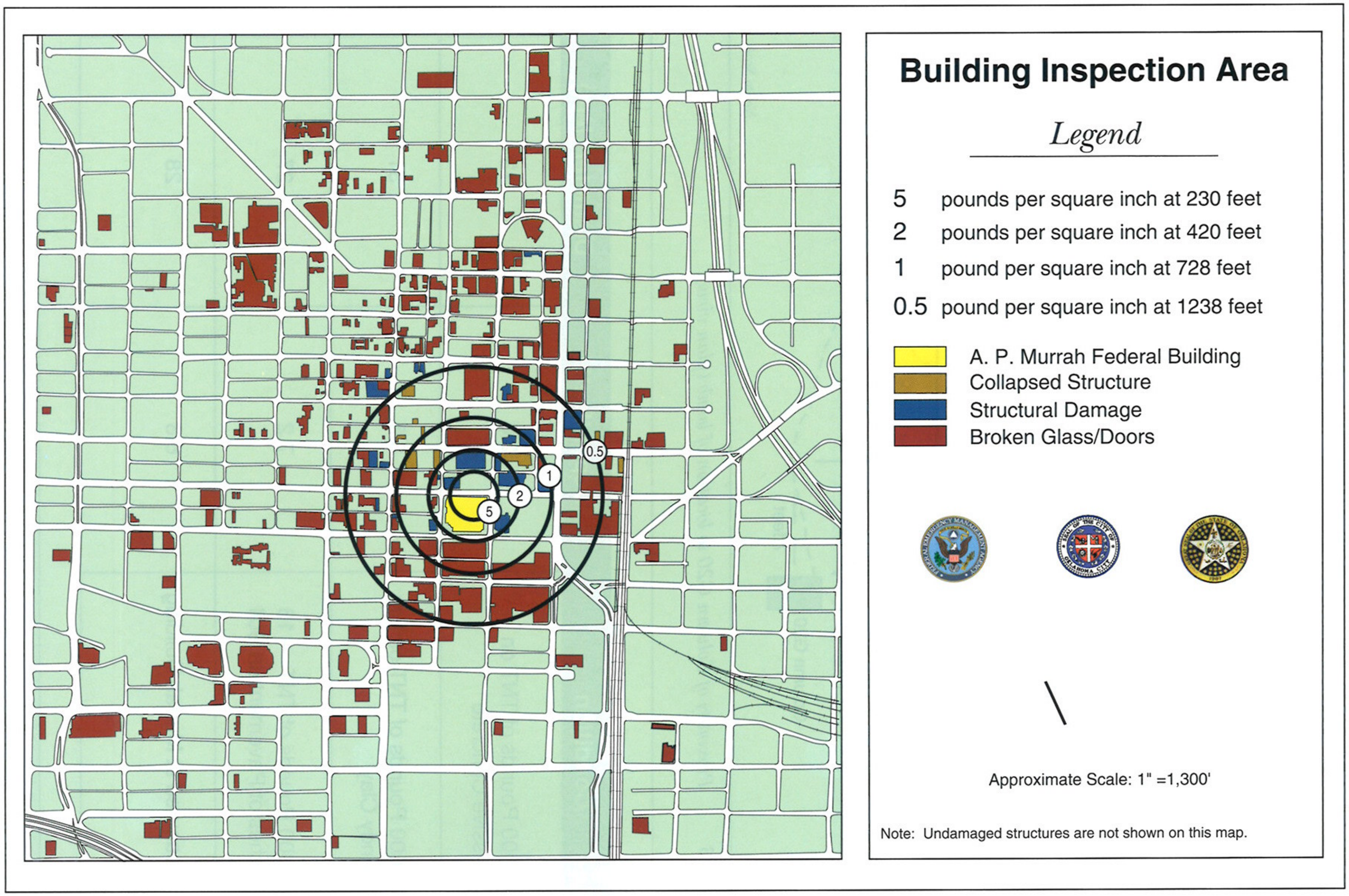


Figure 3-4 *Blast overpressure.*

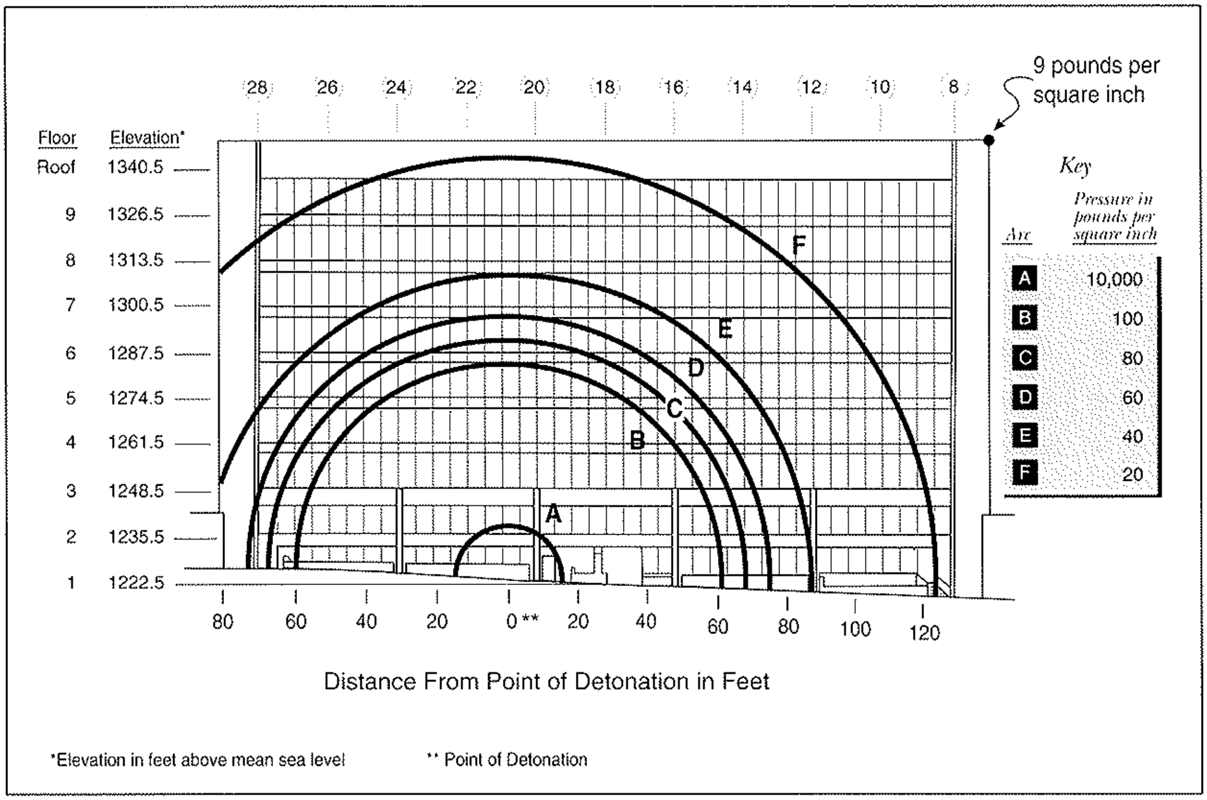


Figure 3-5 *Peak overpressures on north elevation of nine-story portion of Murrah Building.*

Of particular interest is the loading on the nine-story portion of the Murrah Building. The peak overpressures on the north elevation are shown in Figure 3-5. These range from a maximum of over 10,000 pounds per square inch at the point closest to the detonation to a minimum of 9 pounds per square inch at the upper west corner, with an equivalent uniform value of approximately 140 pounds per square inch. While these pressures are extremely large, they act for a limited duration as shown in Figure 3-6. The duration ranges from a maximum in the upper west corner to a minimum at the point closest to the blast and has an equivalent uniform value for a triangular pulse of about 5 milliseconds.

Any explosive detonation near the ground surface causes a ground shock motion in addition to the airblast loading. For the surface detonation of 4,000 pounds of TNT, the free-field motion at the center of the building is shown in Figure 3-7. There is a rapid rise after arrival to a peak value of 9 inches per second, followed by a gradual decay over the next 270 milliseconds In comparison to the extreme airblast loading, this ground shock motion is of little consequence to the structure.

3.2.2 Blast Analysis of Column G20

As shown in Figure 3-3, the bomb crater places the explosive device in close proximity to Column G20. In fact the scaled range is only

$$\mathbf{R/W^{1/3} = 15.6\ ft\ /\ 4000\ lb^{1/3} \quad 1.0\ ft/lb^{1/3}}$$

in which R is the horizontal range in feet (see Figure 3-3) and W is the equivalent charge weight in pounds of TNT.

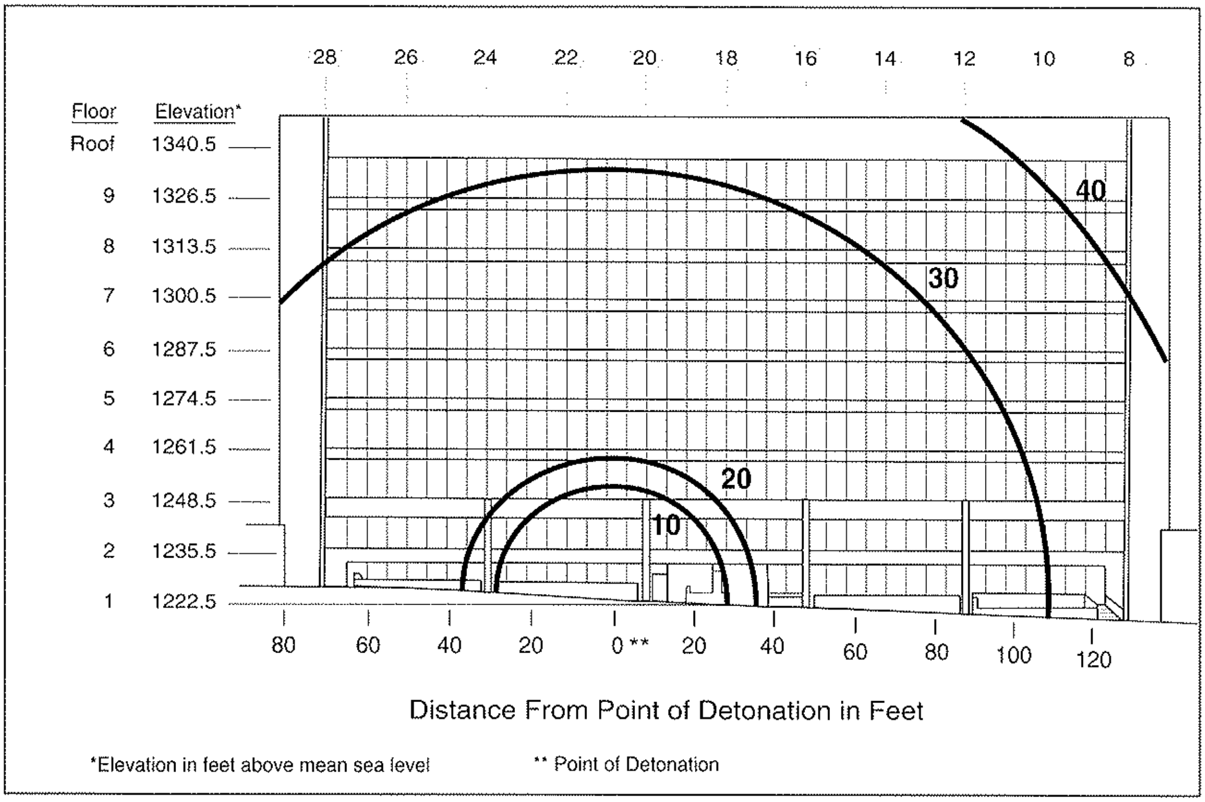


Figure 3-6 *Duration of loading (in milliseconds) on north elevation of nine-story portion of Murrah Building.*

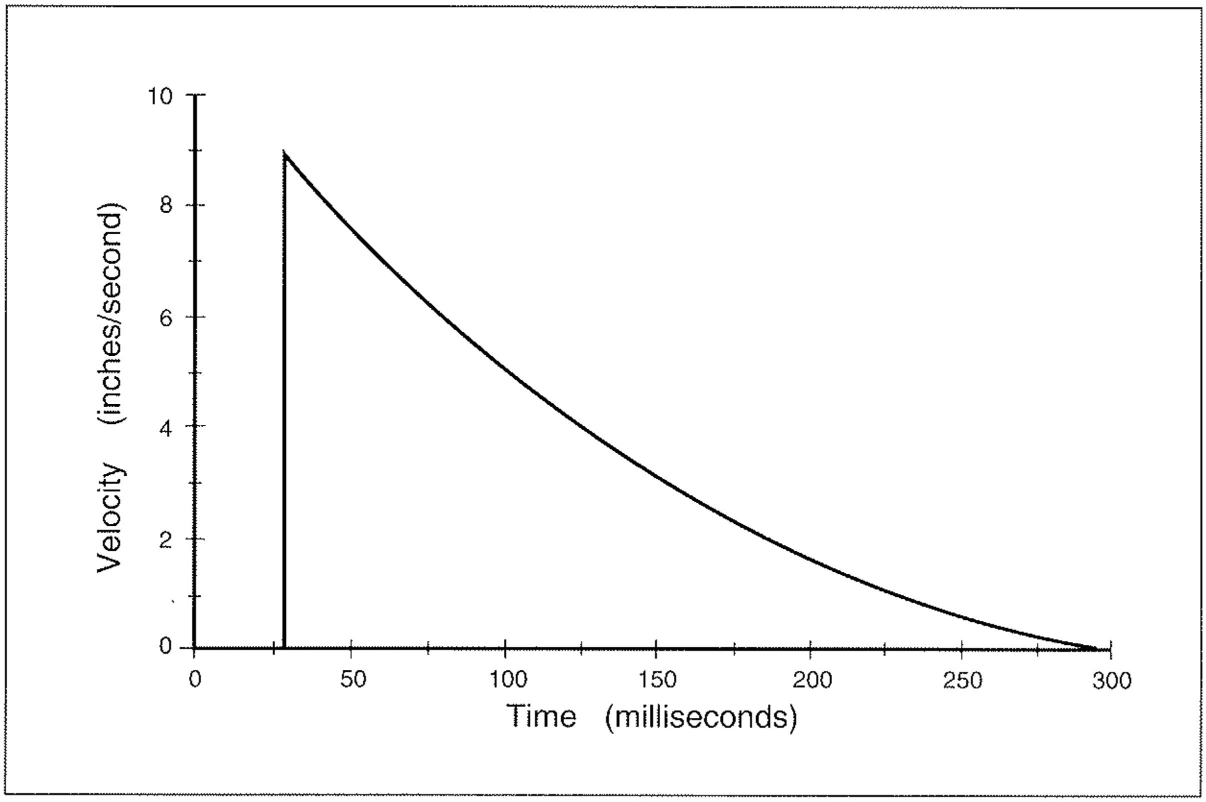


Figure 3-7 *Free field ground shock at center of Murrah Building.*

Based on bomb damage reports from the Second World War (NDRC Summary Technical Report, Reference 19), the destruction of first-story reinforced concrete columns by the brisant effects of blowing out, severing, and undermining occurs at scaled ranges of 3 $ft/lb^{1/3}$from cased charges. From contemporary research (WES-TR-SL-88-22, Reference 18) on the difference between the breaching of reinforced concrete walls by bare and uncased charges, the scaled range for destruction by bare charges is estimated to be 1.5 $ft/lb^{1/3}$. Thus in all likelihood, Column G20 was abruptly removed by brisance. This conclusion is supported by the fact that no one whom the team interviewed found any evidence of this column in the debris or in the crater caused by the explosion.

3.2.3 Blast Analysis of Columns G24, G16, and G12

Column G24 was located outside the range of brisance for uncased charges, but was highly loaded by the detonation. As indicated in Figure 3-8, its response to this load is approximated as a simply supported beam between the first- and third-floor elevations. The column did extend below the first-floor elevation to its supporting caisson, but it was not loaded by the blast below this level and received some support from the surrounding soil. The column was also laterally connected at the second floor by transfer strut, 2B-13, but this feature provided little restraint in the east-west direction excited by the blast.

As indicated in Figure 3-9, the column resisted this loading about its weak axis. This cross section was 36 inches wide by 20 inches deep. It was reinforced with 20 #11 vertical bars and 2 #4 horizontal ties at 16 inches on center. The strength was limited by the shear resistance at the ends of the column. From the material strengths measured in this study and the estimated axial prestress from dead and actual live loadings, this limiting capacity, Vu, corresponds to 52 pounds per square inch uniformly distributed on the 36-inch face.

Figure 3-10 shows the blast loading on Column G24. On the front face, the load rises abruptly to the reflected pressure, 1400 pounds per square inch. When the blast clears this face, it falls to the sum of incident and dynamic pressures. The blast subsequently arrives at the rear face and rises gradually to the sum of the incident and dynamic pressures at this range and orientation. The effective triangular duration of the net loading is only about 1 millisecond.

The response of Column G24 to this loading, illustrated in Figure 3-11, is specifically the lateral deflection of the column at the midpoint, or second-story level, as a function of time measured after the detonation of the bomb. Notice that most of this response occurs after the net load has diminished to zero so that it is an impulsive structural event. When this deflection reaches 1.0 inch, the shear at the supports, V, exceeds Vu and a brittle failure of the element occurs. As the axial prestress and corresponding shear capacity are greater at the first floor than at the third floor, this failure is expected at the top. Immediately after the blast (Figure 3-12), this column was missing from above the first floor, in agreement with the results of this analysis.

As shown in Table 3-2, the slant range (distance from the explosion to the column midheight) is greater for Column G16 than for Column G24. At this distance of 50 feet, the peak pressure is still 641 psi. According to an analysis similar to that performed for Column G24, this loading just reaches the shear capacity. This implies an incipient brittle failure which is consistent with the conditions shown in Figure 3-13.

However, Column G12 did endure the direct blast effects of the bomb. It was located at a slant range of 89 feet as indicated in Table 3-2. Here, the loading was 115 pounds per square inch. The associated response is only 0.1 times the capacity to resist. The results of this analysis are consistent with the intact condition of this column after the bombing, as shown in Figure 3-14.

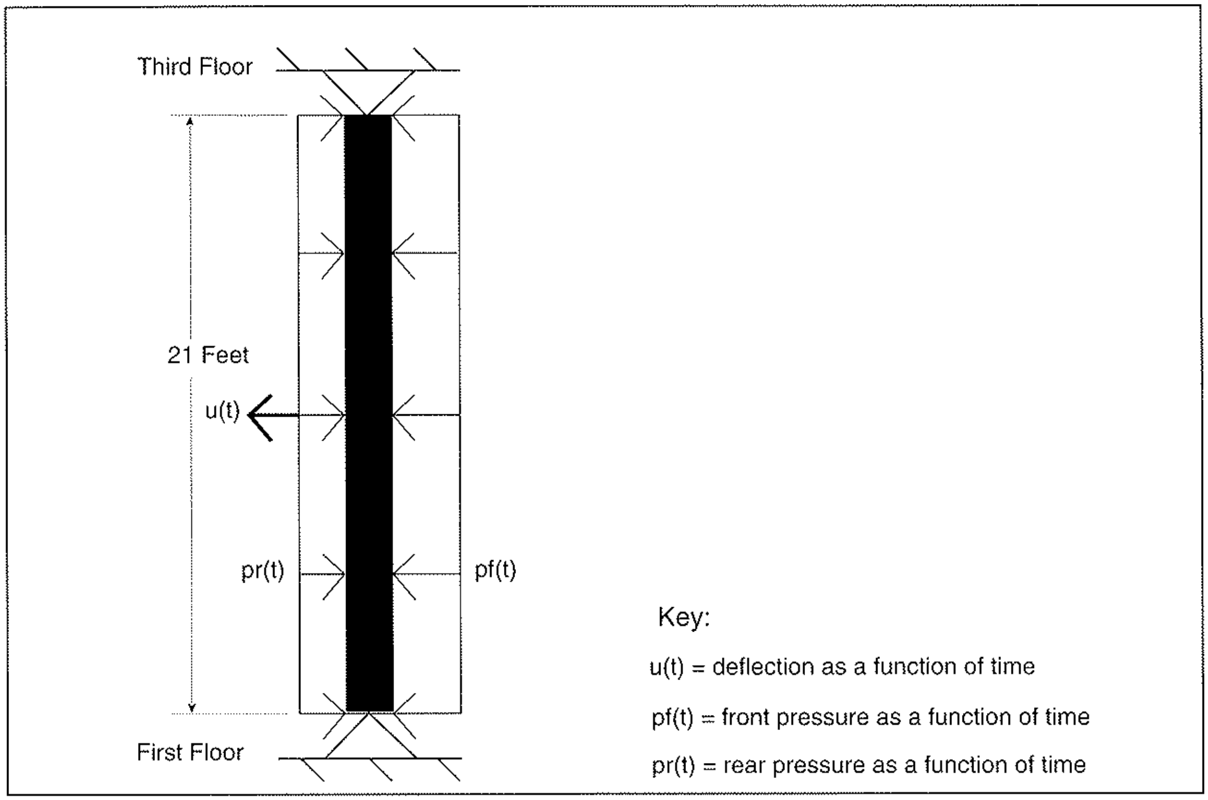


Figure 3-8 *Model of Column G24 (singe degree of freedom).*

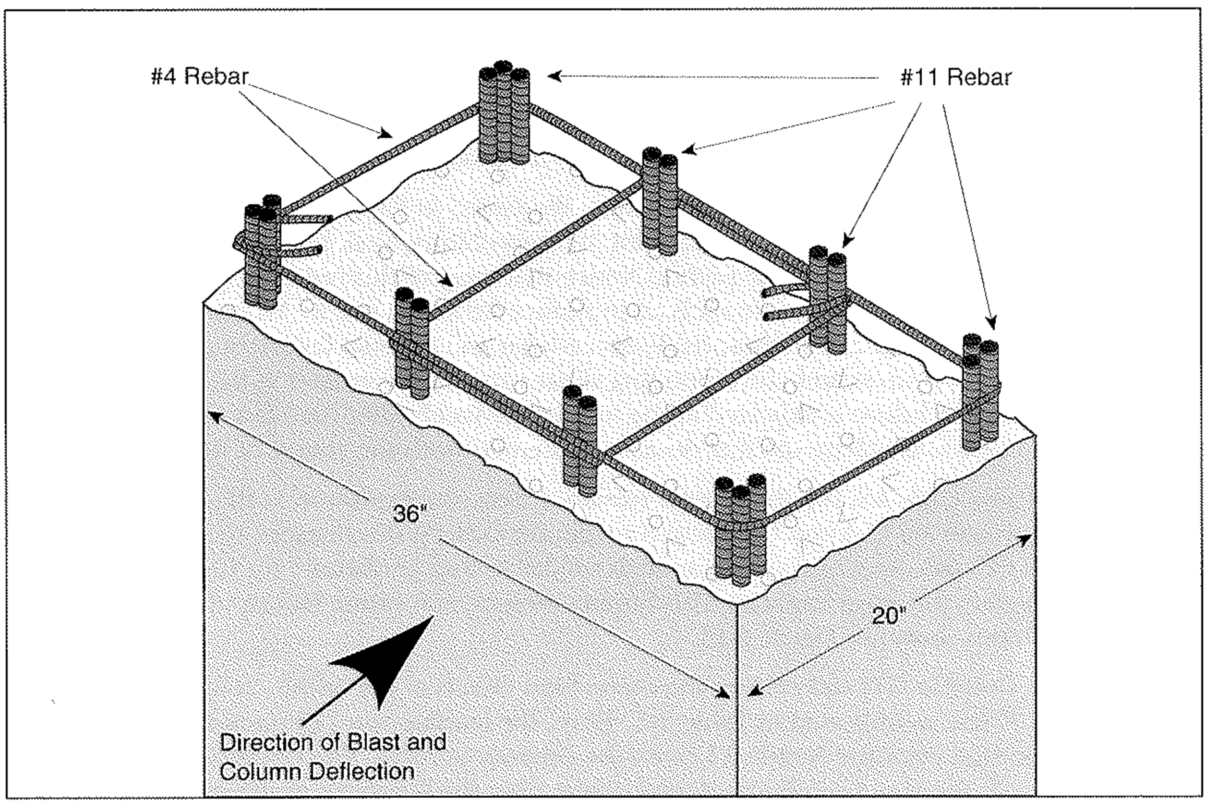


Figure 3-9 *Cross section of Column G24.*

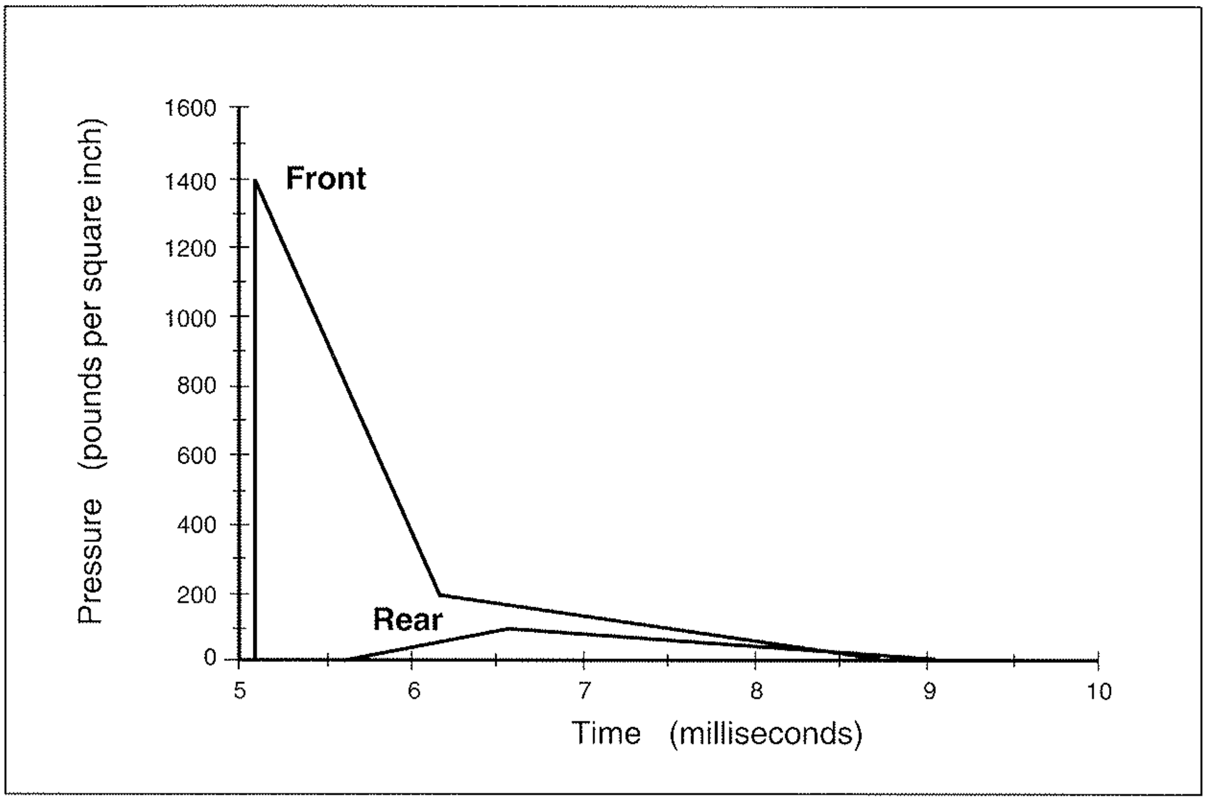


Figure 3-10   Blast loading of Column G24.

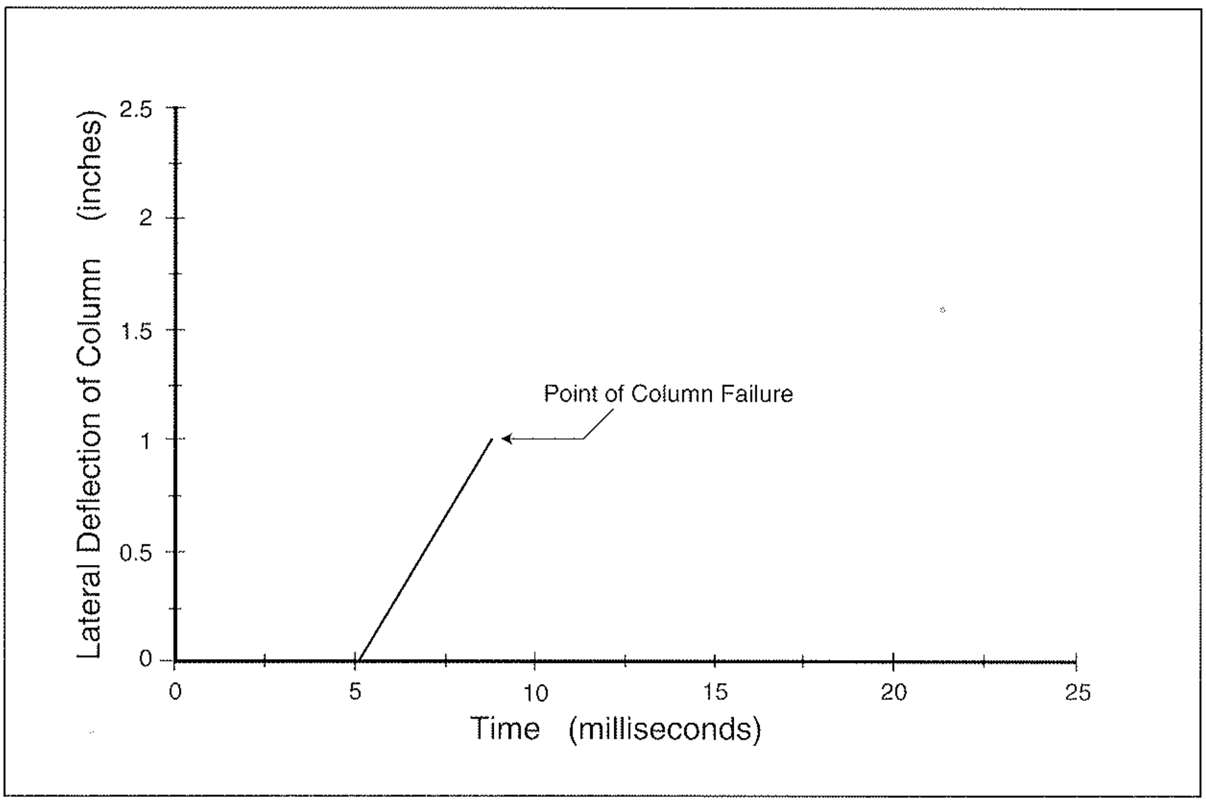


Figure 3-11   Blast response of Column G24 at midpoint (second-floor elevation).

Figure 3-12 Damage at Column Line G.

	Column Numbers			
	G24	G20	G16	G12
Slant Range (feet)	37	21	50	89
Peak Pressure (pounds per square inch)	1,400	5,600	641	115
Duration (milliseconds)	1.3	(1)	1.7	1.4
Deflection (inches)	2.2	(1)	1.2	0.2
Shear at Supports / Limiting Capacity	1.8	(1)	1.0	0.1

(1) Column destroyed by brisance

Table 3-2 Blast response of intermediate columns supporting north transfer girder.

Figure 3-13 View of damage at north face of Murrah Building.

Figure 3-14 Column G12 intact.

3.2.4 BLAST ANALYSIS OF SLABS

The floor slabs in close proximity to the bomb were directly loaded by the blast. The facade of the north elevation consisted of 5-foot by 10-foot glass panels restrained by aluminum channels. This glazing offered insignificant resistance to the propagating blast wave. Upon the failure of the glazing, the blast filled the structural bays above and below each floor slab. The filling pressures below the slab were greater than the filling pressures above and caused an upward load on each slab.

This net upward loading is shown in Figure 3-15 as a spatially uniform pressure for the purposes of this report. The slab is modeled as a simply supported element spanning from east to west between floor or roof beams. The length of this span between the supporting beams is 16 feet.

A cross section of a typical floor slab is shown in Figure 3-16. At mid-span, the slab is 6 inches deep and reinforced with #4 bars at 9 inches on center in the east-west direction and 18 inches on center in north-south direction. Because of their location near the bottom of the slab, these bars provide little resistance to an upward loading, and the capacity is a uniform load of only 0.40 pound per square inch. The roof slabs are typically 6 inches thick as well but are reinforced with #4 bars at 16 inches on center in the east-west direction and therefore provide a resistance of 0.24 pound per square inch at mid-span.

Figure 3-17 shows the loading on the fifth-floor slab between Column Lines 20 and 22, which is considered here for illustration. The loadings are assumed to be the incident overpressures at the range of the midpoint of the structural bays above and below each slab. These loadings are further represented by triangular pulses as shown in Figure 3-17. In this particular case, the load from below has a peak of 154 pounds per square inch, while the load above is only 87 pounds per square inch. In both cases, these loads act for relatively short durations.

The response of this slab to the loading is shown in Figure 3-18. In this calculation, the static dead and actual live loadings are included as well as the blast loading from Figure 3-17. Notice that the upward response of the slab has a long period, and the blast event represents an impulsive loading condition. In this case, the maximum deflection is 9.3 inches. This deflection exceeds the ultimate capacity of the floor slab and also represents a rotation of 5.3 degrees over the 16-foot span. Under these conditions, the collapse of the slab from the direct blast loading is expected.

A similar analysis was performed for the other floor and roof slabs in the building. The results are shown in Figure 3-19. In particular, the floor slabs in the fifth floor and below between Column Lines 18 and 24 were sufficiently loaded by the blast to fail as shown. However, the other slabs responded elastically to the differential blast loading and in some cases were not loaded above the static downward loads.

Figure 3-20 approximates the inward extent of this directly induced slab failure. It assumes that the distance of blast propagation is the range to determine the net upward loading. The results show a penetration of failure to 40 feet at the second floor, which diminishes to zero at the sixth floor.

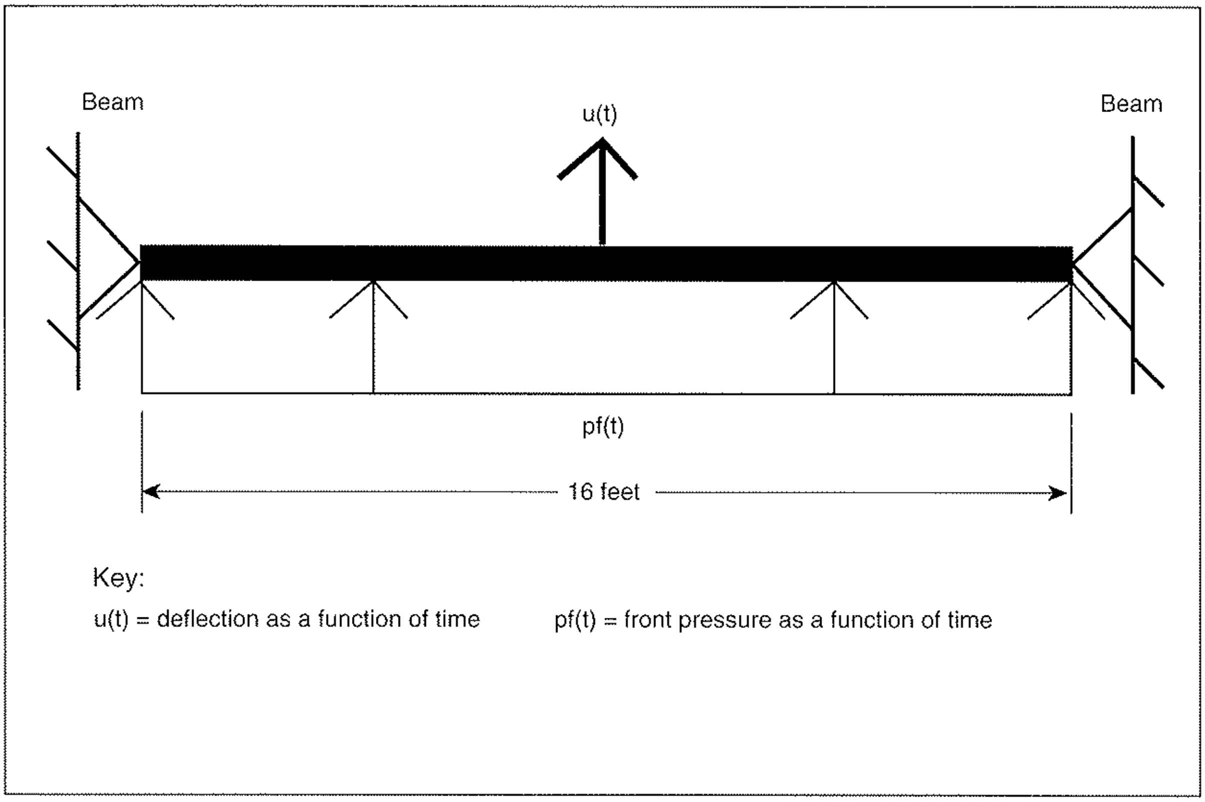


Figure 3-15 Model of slab (single degree of freedom).

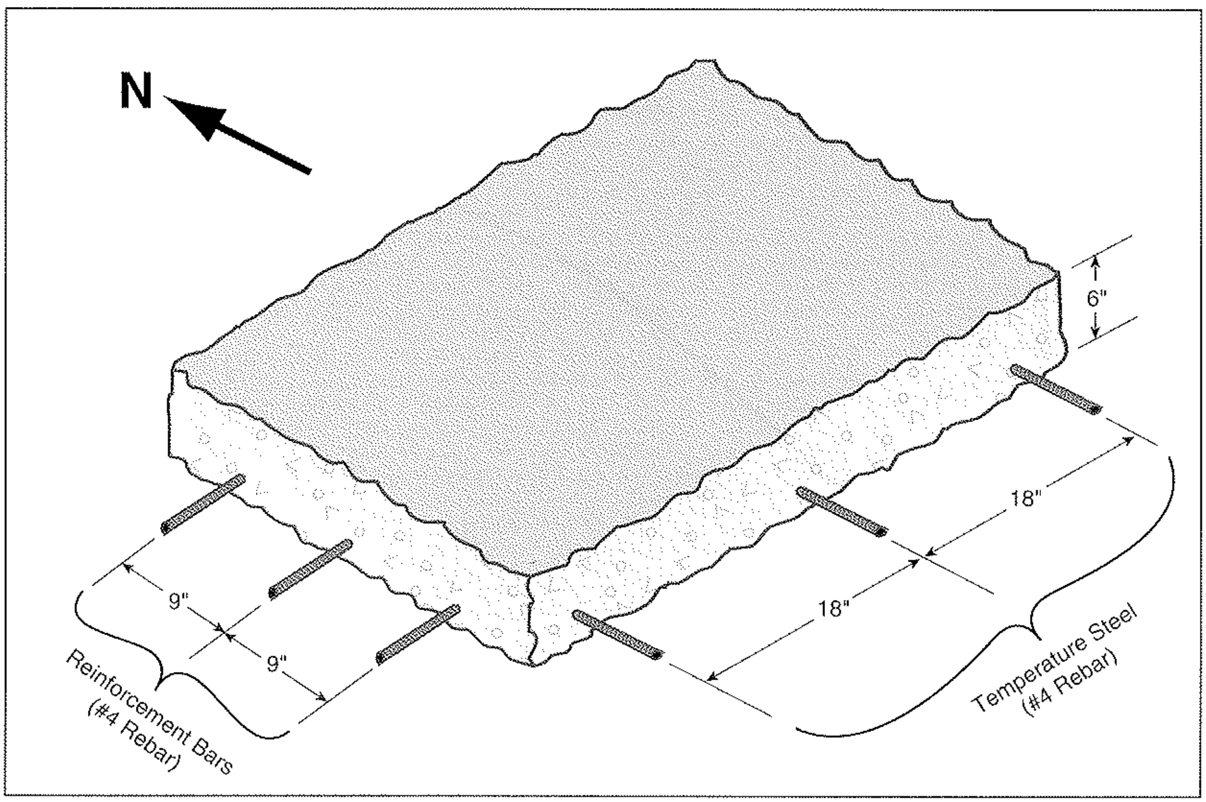


Figure 3-16 Typical section of floor slab at centerline of span.

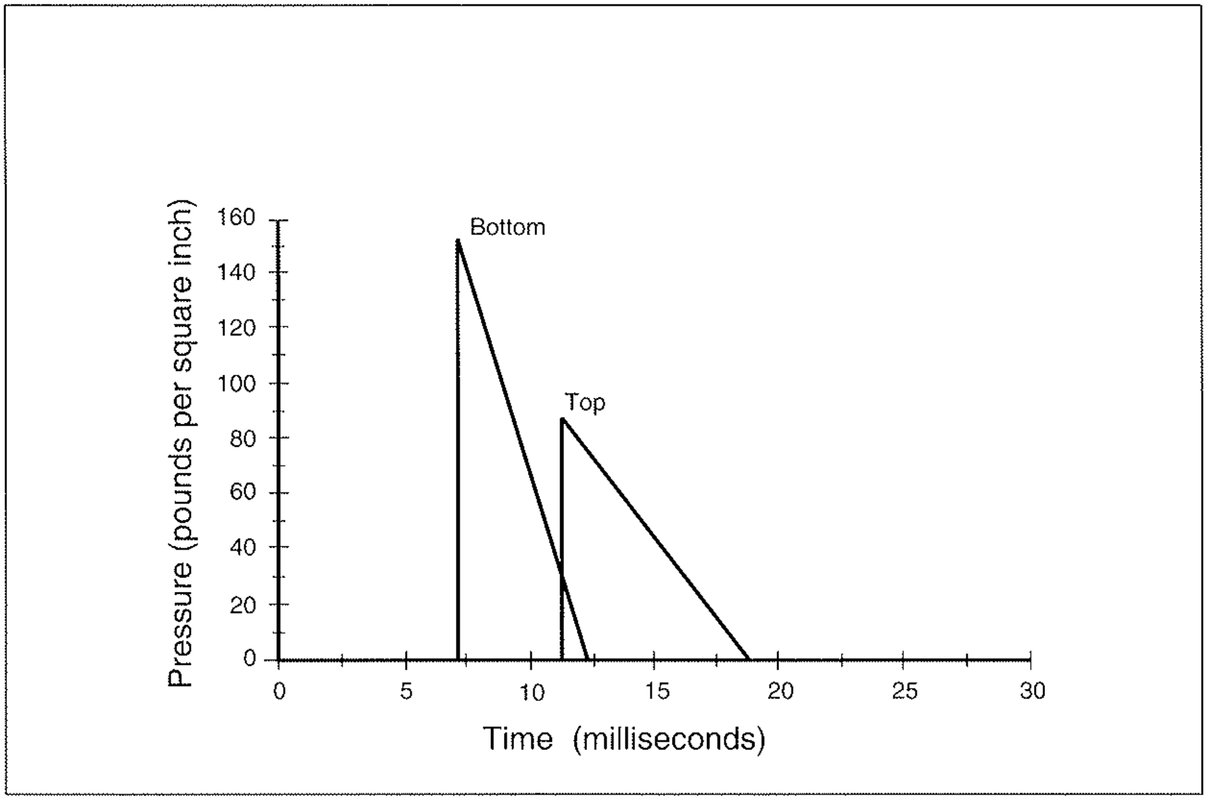


Figure 3-17 *Blast loading of fifth-floor slab between Column Lines 20 and 22.*

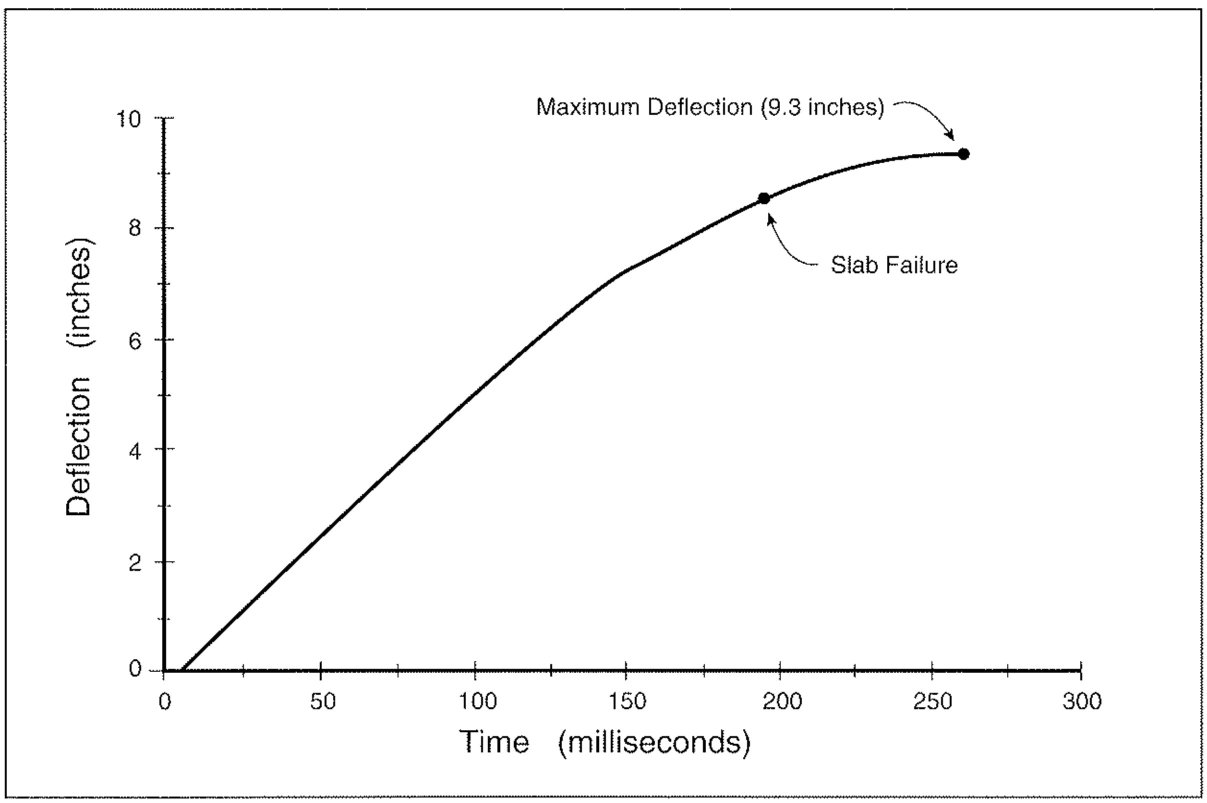


Figure 3-18 *Blast response of fifth-floor slab between Column Lines 20 and 22.*

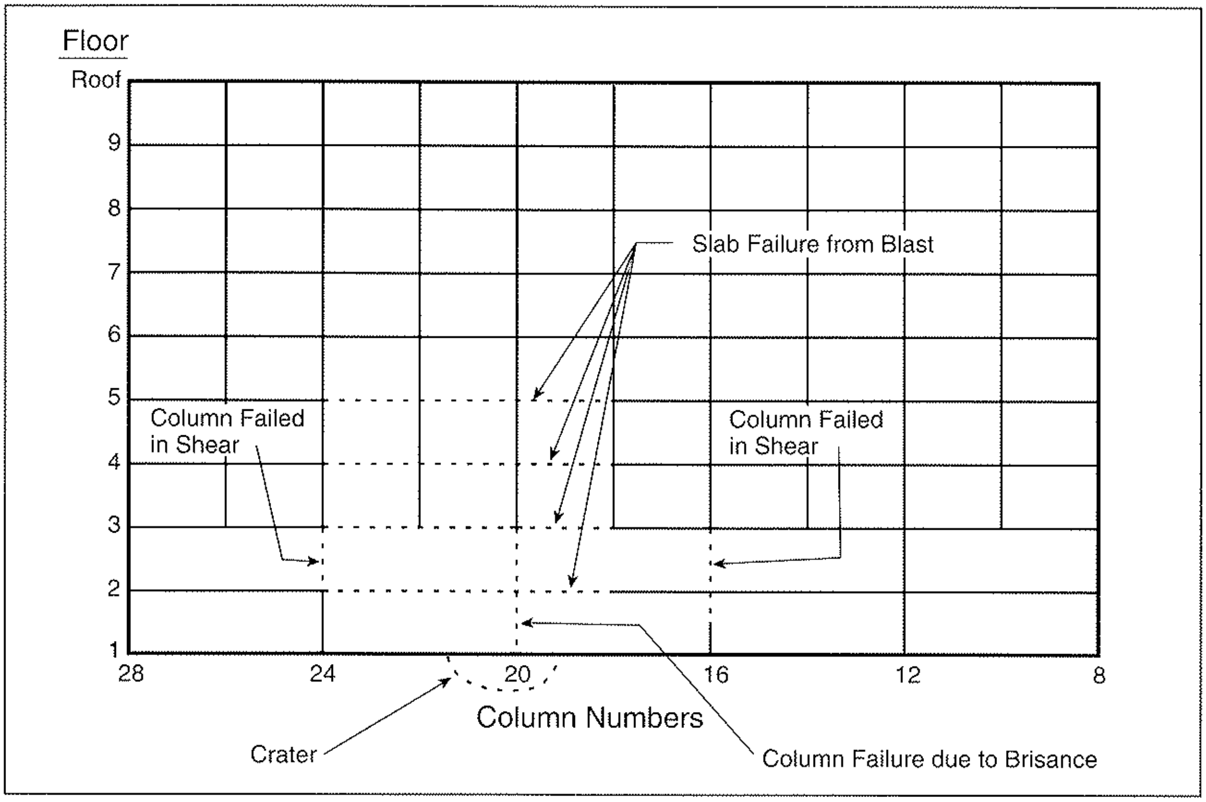


Figure 3-19 *Schematic of blast response, north elevation of nine-story portion of Murrah Building at Column Line G.*

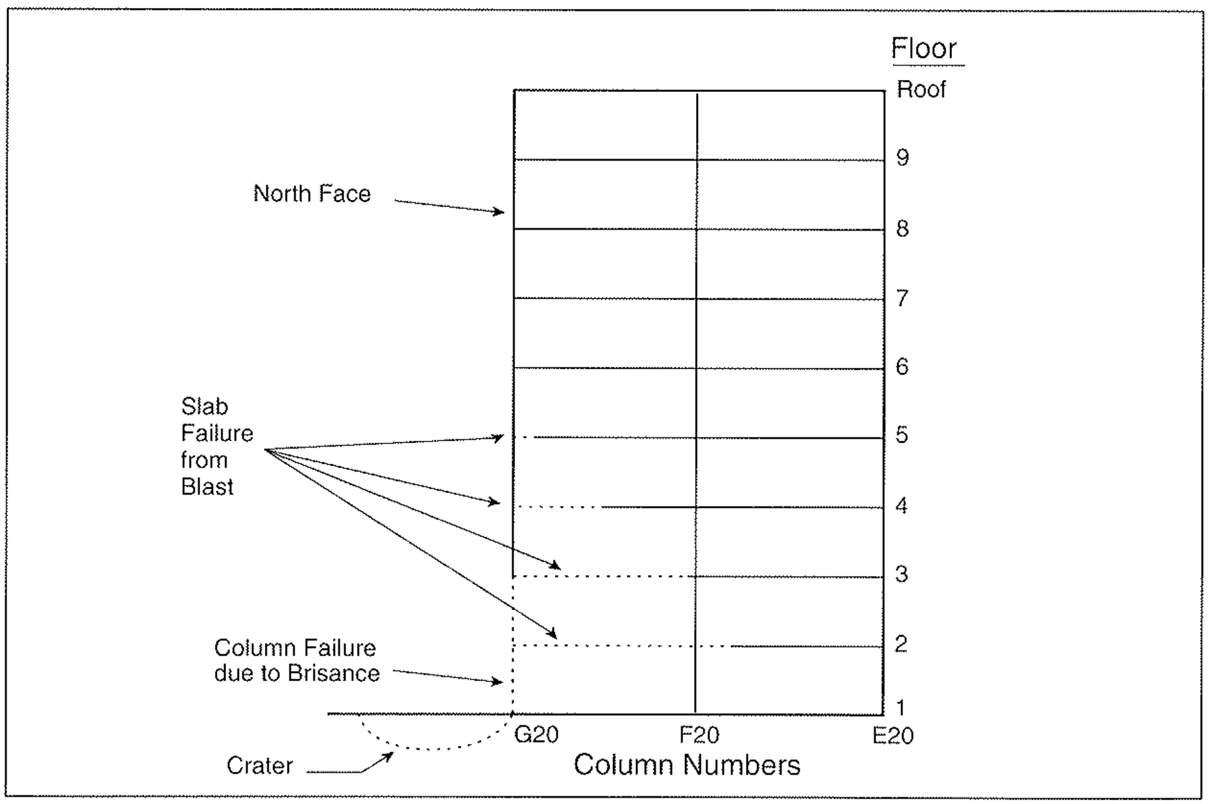


Figure 3-20 *Cross section of nine-story portion of Murrah Building showing blast response of slabs.*

3.3 Possible Mechanisms

3.3.1 Introduction

This section summarizes the study of the structural failure mechanism for the nine-story monolithic reinforced concrete portion of the Murrah Building. The study was carried out to quantify the reasons for the observed failures and to develop options for feasible changes in detail that would reduce the probability of damage from blast and seismic loading in other Federal buildings.

All structural data used in the evaluation have been obtained from the structural drawings (Nos. 7-1, 7-2, and 7 -26 through 7-40) provided by the Engineer of Record for the building, Mr. Kirkpatrick. Nominal material strengths are based on those documented in Drawing No. 7-1 by Mr. Kirkpatrick. The design strengths are recorded to be 4,000 pounds per square inch for the concrete and 60,000 pounds per square inch for the reinforcement. Tension tests of two samples of #8 reinforcing bars taken from the Murrah Building indicated yield stresses of approximately 72,000 and 77,000 pounds per square inch. Tests of three concrete cores from the building indicated compressive strengths of 4,180, 6,550, and 5,540 pounds per square inch. These correspond to design compressive strengths of 4920, 7710, and 6520 pounds per square inch.

The following sections describe the structural dimensions, present the calculated nominal strengths, describe the types of calculations carried out, and discuss the inferences from the analyses.

3.3.2 Structural Dimensions

Figures 3-21 through 3-23 show the column line designations, typical column cross-sectional dimensions, and spans for the nine-story portion of the Murrah Building. Information about the approximate size and location of the crater caused by the blast is provided in Figures 3-2 and 3-3, earlier in this section.

The arrangement of reinforcing bars in the 3-foot-wide by 5-foot-deep transfer girder on Column Line G at the third floor is summarized in Figure 3-24. The girder is used to pick up alternate columns and change column spacing from 20 feet in upper stories to 40 feet in lower stories. Reinforcing bar arrangements for the spandrel girders on Column Line G at the fourth through ninth floors are shown in Figure 3-25. Reinforcing bar arrangements for the roof girder on Column Line G are shown in Figure 3-26. Information in the available drawings about the length of the top reinforcing bars for the roof girder was vague.

Cross-sectional distributions of the girder reinforcing bars are shown in Figure 3-27.

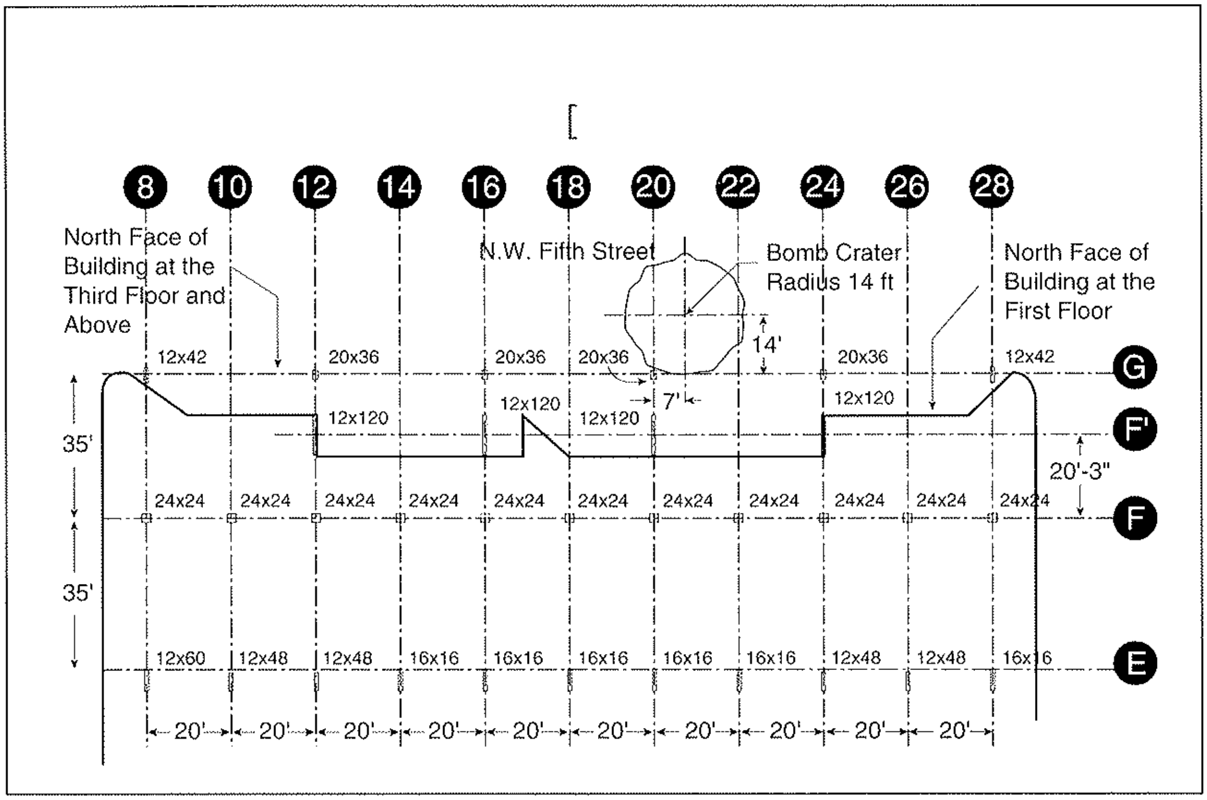


Figure 3-21   Column locations and dimensions – first floor.

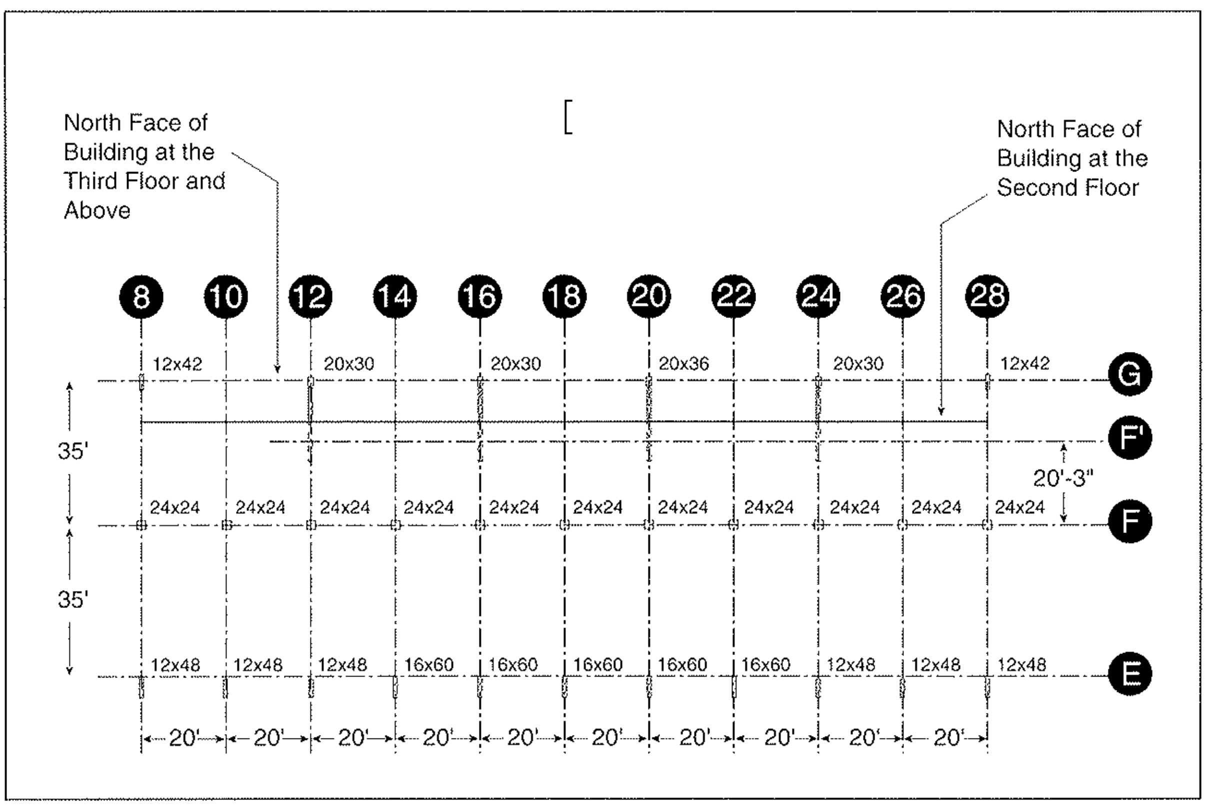


Figure 3-22   Column locations and dimensions – second floor.

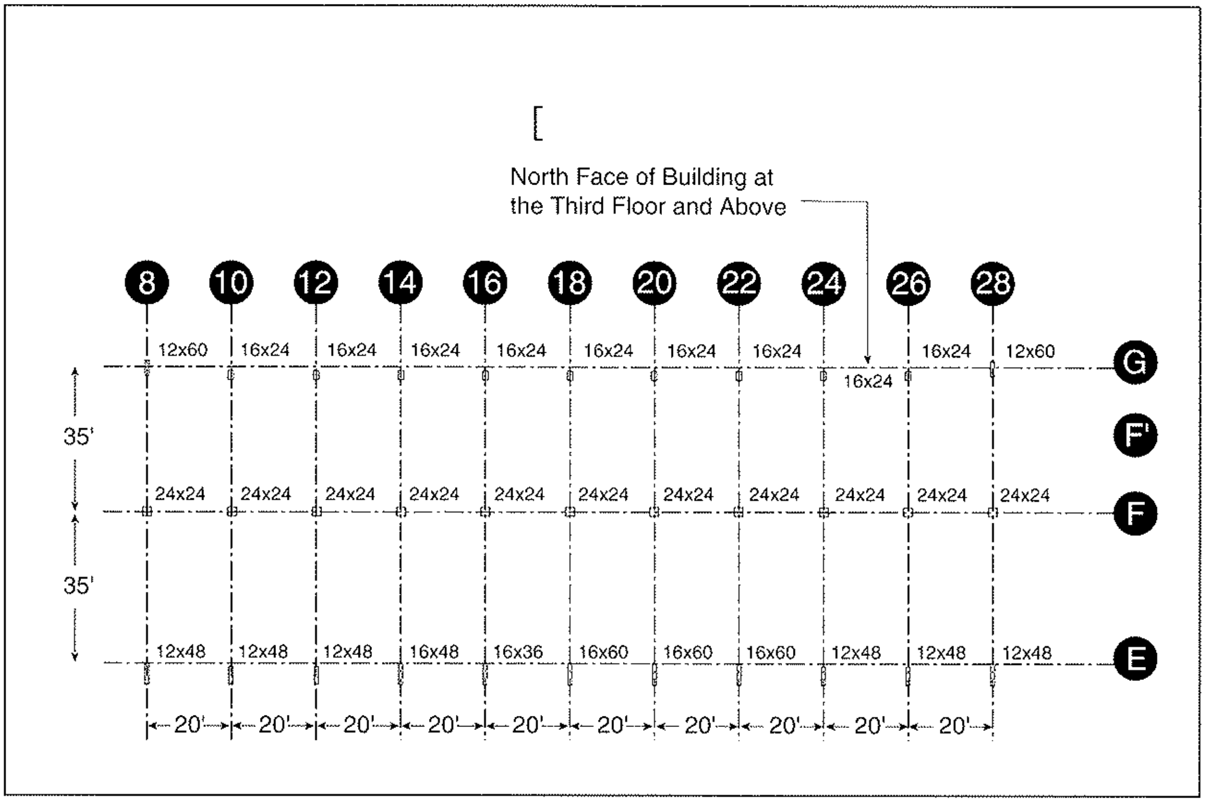


Figure 3-23 *Column locations and dimensions – third floor through roof.*

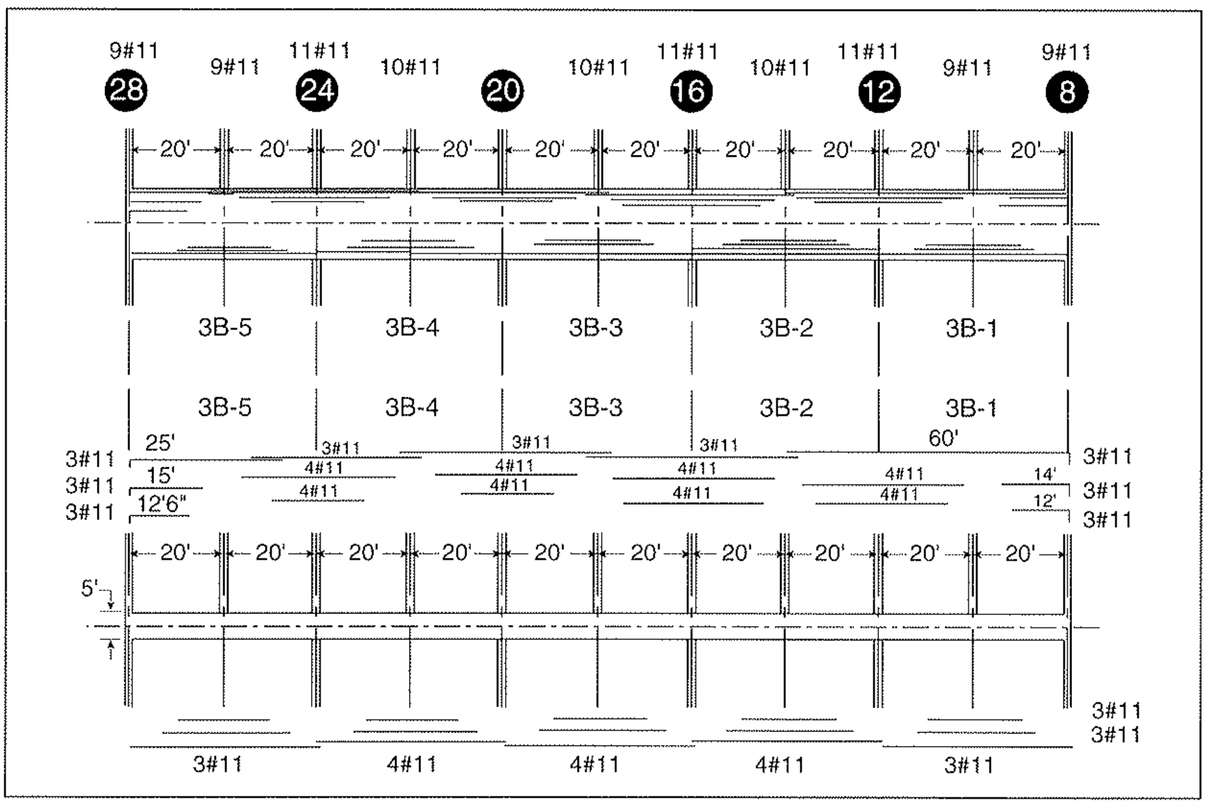


Figure 3-24 *Schematic of reinforcement for transfer girder on third floor at Column Line G.*

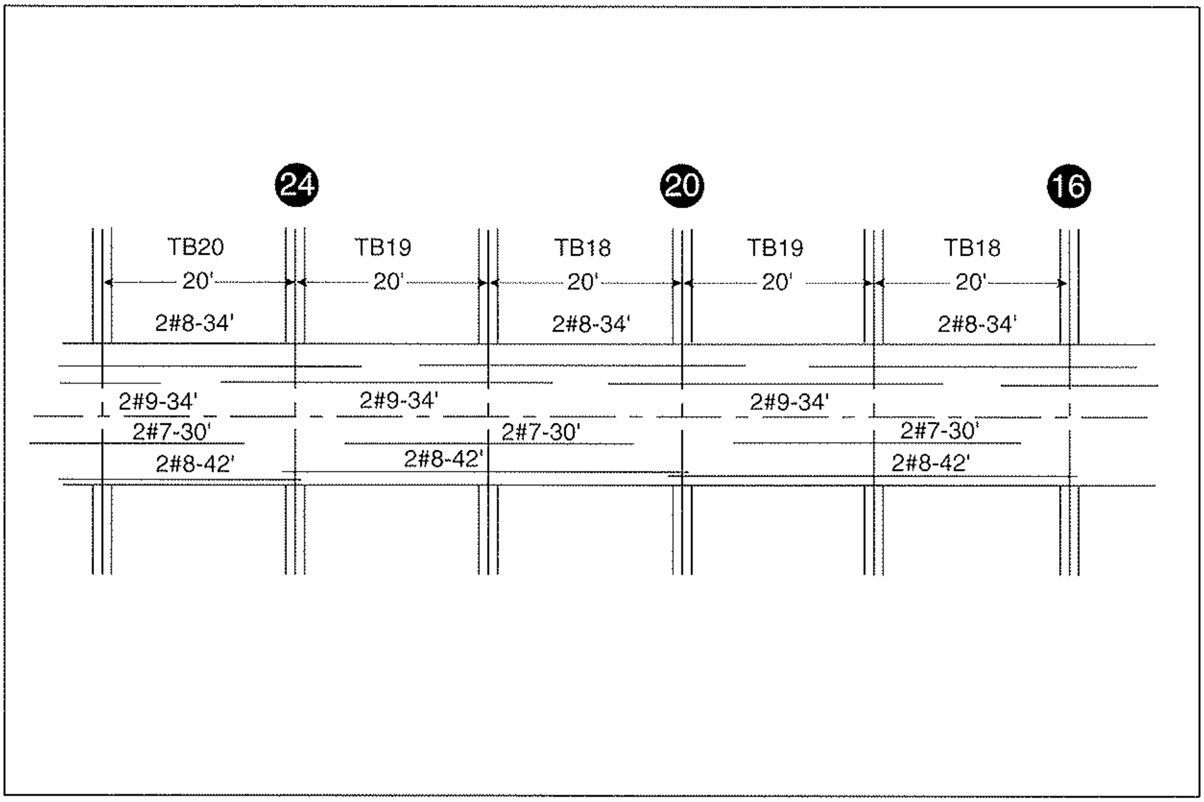


Figure 3-25 *Schematic of distorted section showing reinforcing bar arrangement for spandrel girders on fourth through ninth floors at Column Line G.*

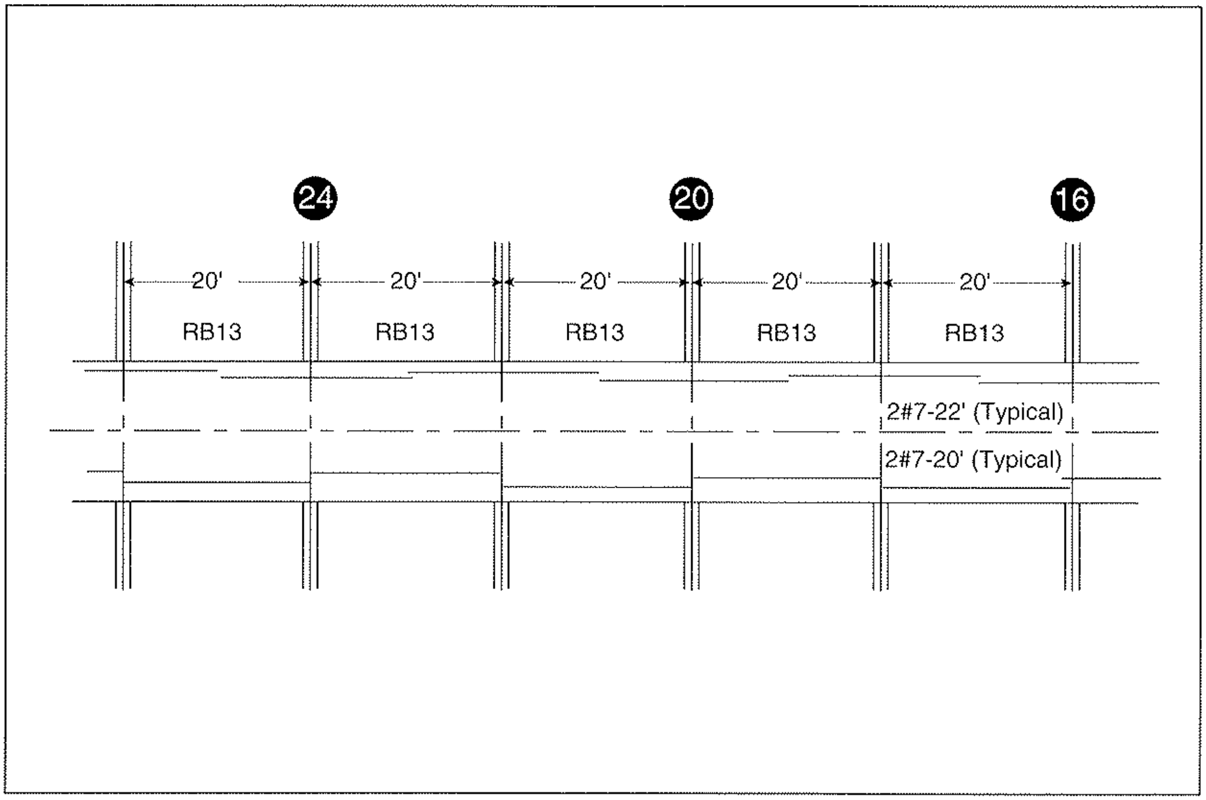


Figure 3-26 *Schematic of distorted section showing reinforcing bar arrangement for roof girder at Column Line G.*

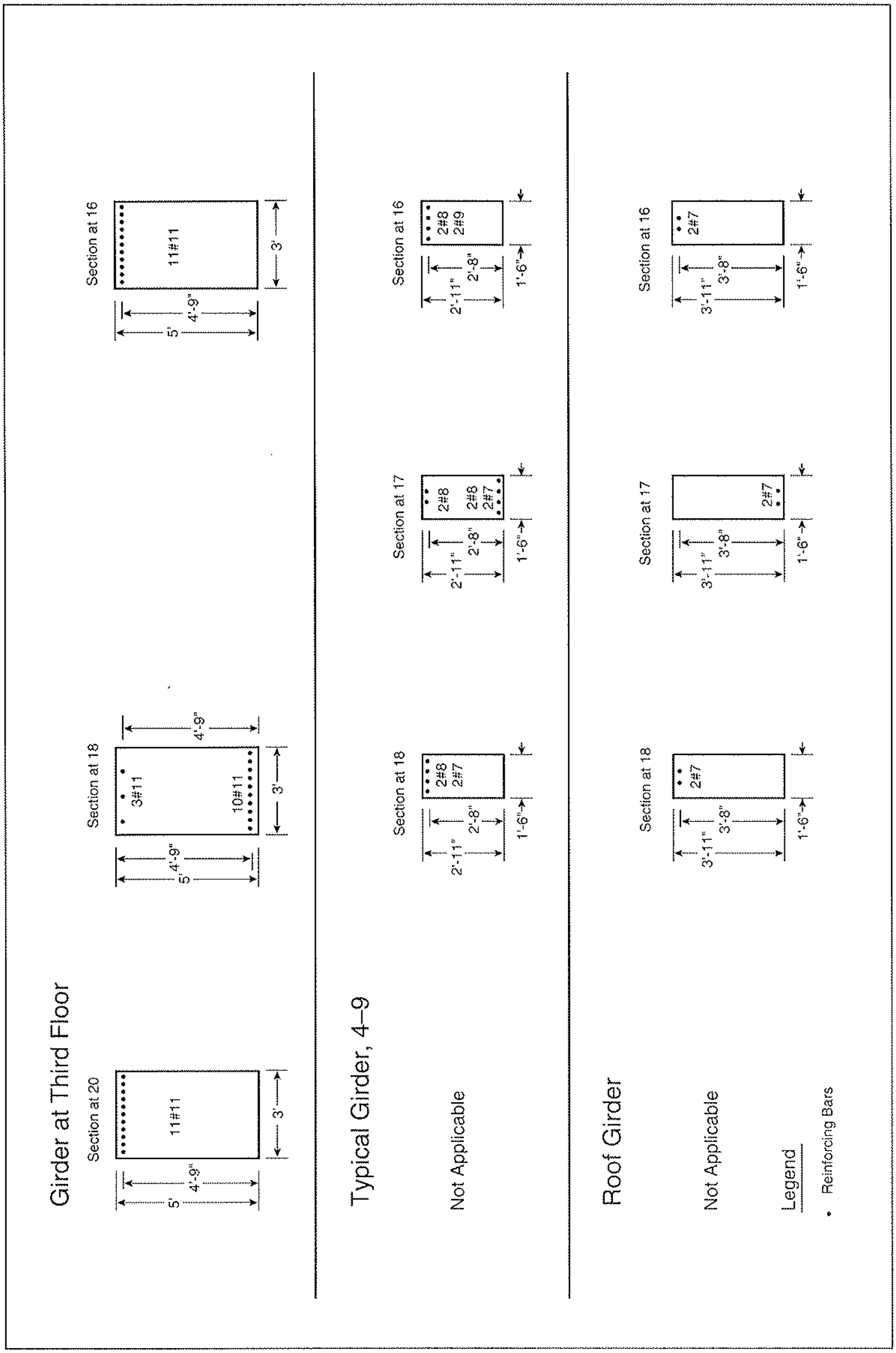


Figure 3-27 Cross-sectional distribution of girder reinforcing bars.

3.3.3 Calculated Nominal Section Strengths

Tables 3-3 through 3-7 contain the nominal flexural strengths of the girder sections and the data used in the calculations. The flexural strength, M_u, was based on the expression:

$$M_u = A_s f_{sy}\, d(1\text{-}.5\ \frac{\rho f_{sy}}{0.85 f'_c})$$

Where:
M_u = Flexural moment strength of a singly reinforced section
A_s = Total cross-sectional area of tensile reinforcement
f_{sy} = Yield strength of the tensile reinforcement
d = Distance from top fiber in compression to centroid of tensile reinforcement
ρ = Tensile reinforcement ratio (A_s/bd, where b = width of girder)
f'_c = Compressive strength of the concrete

Interactions of axial load and bending moment capacity calculated for Column G20 (Figure 3-28) are shown in Figure 3-29.

	Section 16	Section 18	Section 18	Section 20
Width (inches)	36	36	36	36
Effective Depth (inches)	57	57	57	57
Number of Bars	11	10	3	11
Bar Area (square inches)	1.56	1.56	1.56	1.56
Number of Bars	—	—	—	—
Bar Area (square inches)	—	—	—	—
Sum Area (square inches)	17.2	15.6	4.7	17.2
Reinforcement Ratio	0.0084	0.0076	0.0023	0.0084
Flexural Moment Strength (thousands of pound-inches)	54,357	49,773	15,684	54,357
Flexural Moment Strength (thousands of pound-feet)	4,530	4,148	1,307	4,530
Rounded Flexural Moment Strength (thousands of pound-feet)	**4,530** Negative	**4,150** Positive	**1,310** Negative	**4,530** Negative

Note: Concrete Design Strength = 4,000 pounds per square inch Steel Design Yield Stress = 60,000 pounds per square inch

Table 3-3 Calculated flexural strengths – girder at third floor (steel design yeild stress).

	Section 16	Section 17	Section 17	Section 20
Width (inches)	18	18	18	18
Effective Depth (inches)	32	32	32	32
Number of Bars	2	2	2	2
Bar Area (square inches)	0.79	0.79	0.79	0.79
Number of Bars	2	2	—	2
Bar Area (square inches)	1.00	0.6	—	1.00
Sum Area (square inches)	3.6	2.8	1.6	3.6
Reinforcement Ratio	0.0062	0.0048	0.0027	0.0062
Flexural Moment Strength (thousands of pound-inches)	6,497	5,110	2,960	6,497
Flexural Moment Strength (thousands of pound-feet)	541	426	247	541
Rounded Flexural Moment Strength (thousands of pound-feet)	**540** Negative	**430** Positive	**250** Negative	**540** Negative

Note: Concrete Design Strength = 4,000 pounds per square inch   Steel Design Yield Stress = 60,000 pounds per square inch

Table 3-4   *Calculated flexural strengths – girders at fourth through ninth floors (steel design yeild stress).*

	Section 16	Section 17	Reduced 17	Section 20
Width (inches)	18	18	18	18
Effective Depth (inches)	44	44	44	44
Number of Bars	2	2	—	2
Bar Area (square inches)	0.6	0.6	—	0.6
Number of Bars	—	—	—	—
Bar Area (square inches)	—	—	—	—
Sum Area (square inches)	1.2	1.2	0.0	1.2
Reinforcement Ratio	0.0015	0.0015	0.0000	0.0015
Flexural Moment Strength (thousands of pound-inches)	3,126	3,126	0	3,126
Flexural Moment Strength (thousands of pound-feet)	260	260	0	260
Rounded Flexural Moment Strength (thousands of pound-feet)	**260** Negative	**260** Positive	**0** Negative	**260** Negative

Note: Concrete Design Strength = 4,000 pounds per square inch   Steel Design Yield Stress = 60,000 pounds per square inch

Table 3-5   *Calculated flexural strengths – girder at roof (steel design yeild stress).*

	Section 16	Section 17	Reduced 17	Section 20
Width (inches)	18	18	18	18
Effective Depth (inches)	32	32	32	32
Number of Bars	2	2	2	2
Bar Area (square inches)	0.79	0.79	0.79	0.79
Number of Bars	2	2	—	2
Bar Area (square inches)	1.00	0.6	—	1.00
Sum Area (square inches)	3.6	2.8	1.6	3.6
Reinforcement Ratio	0.0062	0.0048	0.0027	0.0062
Flexural Moment Strength (thousands of pound-inches)	7,506	5,918	3,439	7,506
Flexural Moment Strength (thousands of pound-feet)	626	493	287	626
Rounded Flexural Moment Strength (thousands of pound-feet)	**630** Negative	**490** Positive	**290** Negative	**630** Negative

Note: Concrete Design Strength = 4,000 pounds per square inch Steel Measured Yield Stress = 70,000 pounds per square inch

Table 3-6 Calculated flexural strengths – girders at fourth through ninth floors (steel measured yield stress)

	Section 16	Section 17	Reduced 17	Section 20
Width (inches)	18	18	18	18
Effective Depth (inches)	44	44	44	44
Number of Bars	2	2	—	2
Bar Area (square inches)	0.6	0.6	—	0.6
Number of Bars	—	—	—	—
Bar Area (square inches)	—	—	—	—
Sum Area (square inches)	1.2	1.2	0.0	1.2
Reinforcement Ratio	0.0015	0.0015	0.0000	0.0015
Flexural Moment Strength (thousands of pound-inches)	3,638	3,638	0	3,638
Flexural Moment Strength (thousands of pound-feet)	303	303	0	303
Rounded Flexural Moment Strength (thousands of pound-feet)	**300** Positive	**300** Positive	**0** Positive	**300** Positive

Note: Concrete Design Strength = 4,000 pounds per square inch Steel Measured Yield Stress = 70,000 pounds per square inch

Table 3-7 Calculated flexural strengths – girder at roof (steel measured yield stress).

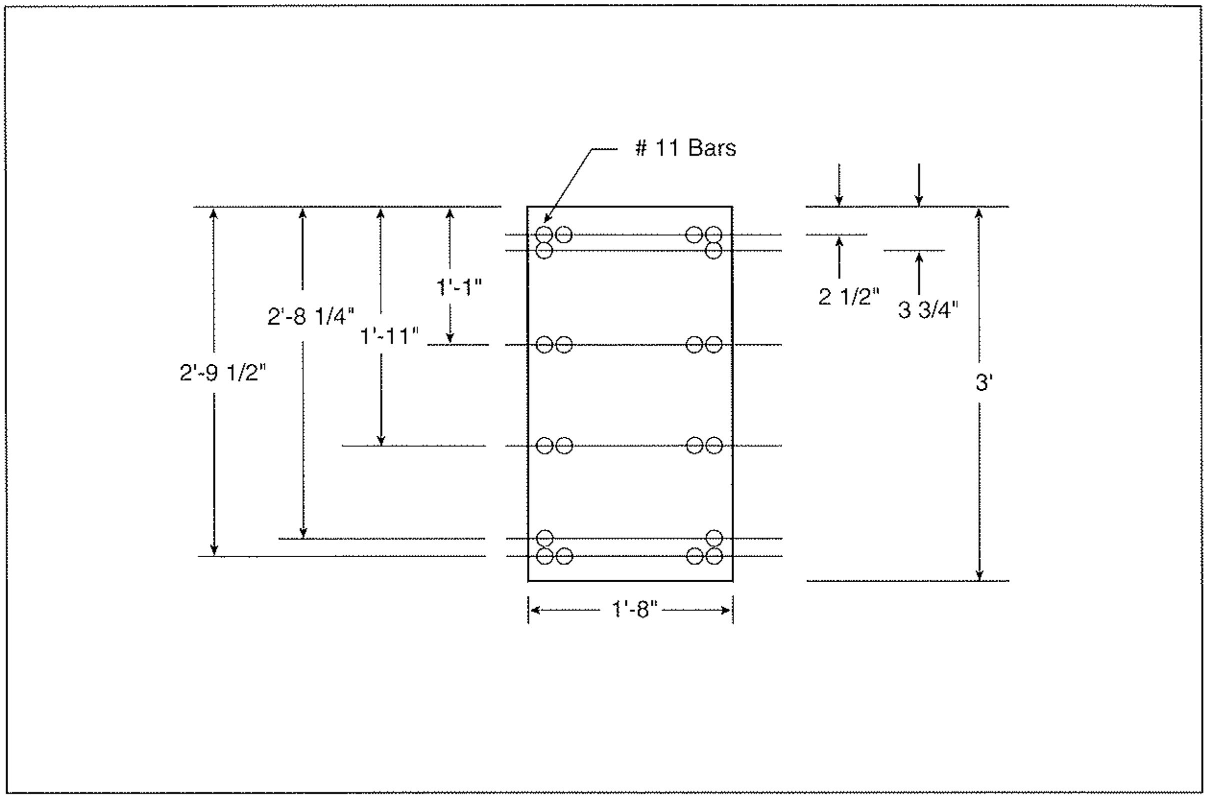


Figure 3-28 *Cross section of Column G20 at first floor.*

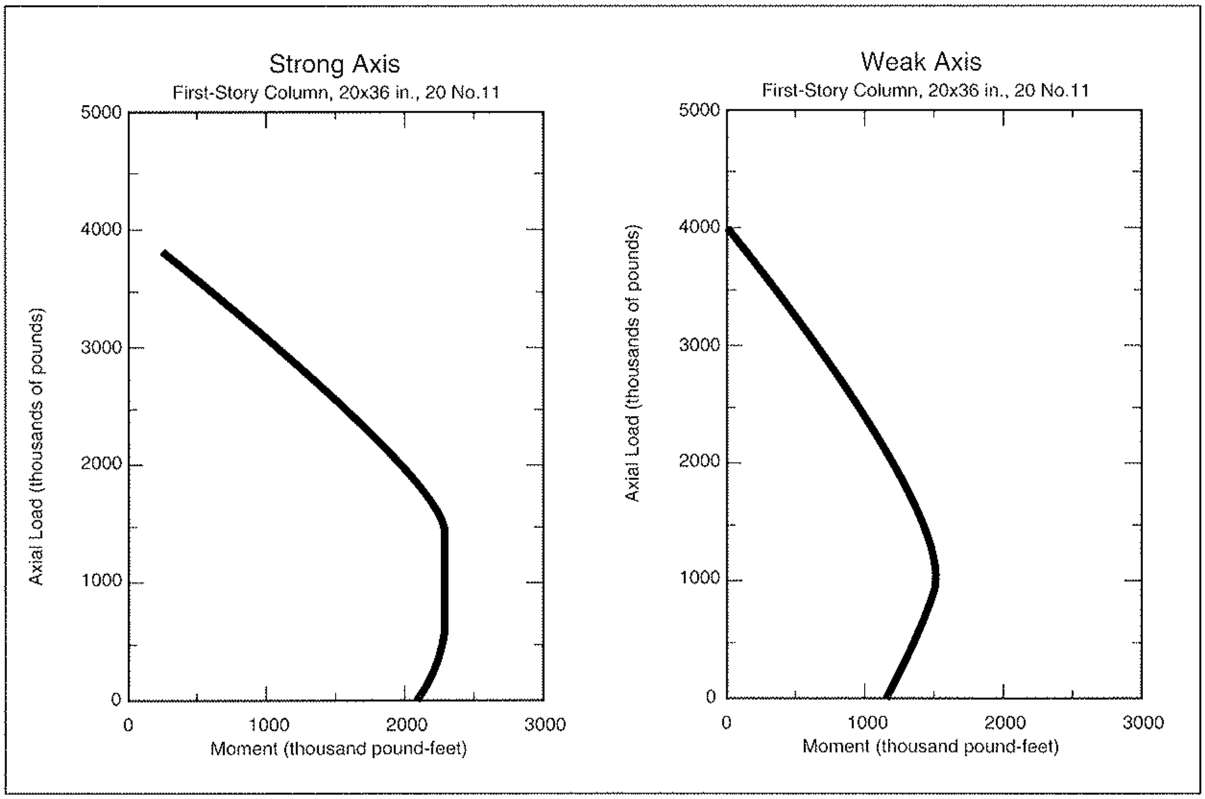


Figure 3-29 *Axial load and bending moment capacity for Column G20.*

One set of calculations (Tables 3-3 through 3-5) was made on the basis of design strengths for the materials without strength reduction factors. For girders with #8 and #7 reinforcing bars, a second set of calculations (Tables 3-6 and 3-7) was made based on a yield stress of 70,000 pounds per square inch on the premise that the measured yield stresses in bars of that size were credible and provided reasonable evidence to support a yield stress of 70,000 pounds per square inch (see Table 2.2). However, the concrete strength and the yield stress of #11 reinforcing bars were assumed to be equal to the design values. Use of the higher values determined from testing would not significantly alter the results.

3.3.4 Limiting Strengths for Column Line G

Three collapse mechanisms for gravity loading of Column Line G were considered. Calculations were made assuming Column Line G to be two-dimensional with a width of 20 feet between columns (tributary width). Effects of in-plane forces on strength were not considered. The three mechanisms are shown in Figure 3-30.

Mechanism 1 represents a typical collapse mechanism for the interior spans of Column Line G in the as-built condition. It is assumed that moment capacity at Column Line 22 is finite at the third through the ninth floors because the bottom-flange reinforcement is understood to be continuous from the information in the drawings. The moment capacities of the girders in the fourth through ninth floors and the roof are assumed to be zero at Column Line 22 because the bottom-flange reinforcement is terminated at support center.

Mechanism 2 is admissible if the first-story section of Column G20 is removed. Positive-moment capacity at support at Column Line 20 is assumed to be zero at all floors because bottom-flange reinforcement is discontinuous at Column Line 20.

Mechanism 3 is a check of the possibility of a reduction in strength (with respect to Mechanism 2) related to discontinuities in the top reinforcing bars. As was done for Mechanism 2, the first-story section of Column G20 is assumed to have been removed. Reduced negative-moment capacities are assumed at Column Lines 18 and 22. Positive-moment capacities at Column Line 20 are set at zero because of the total discontinuity of the reinforcement at that line.

As indicated in Tables 3-8 through 3-13, the calculated unit strengths for the three Mechanisms are as follows:

	Unit Strength (pounds per square inch)	
	Option 1	Option 2
Mechanism 1	490	530
Mechanism 2	60	70
Mechanism 3	100	110

Option 1 (Tables 3-8 through 3-10) refers to all strengths set at nominal design values. Option 2 (Tables 3-11 through 3-13) refers to yield stresses of the #7 and #8 reinforcing bars set at 70,000 pounds per square inch and yield stresses of the #11 reinforcing bars and concrete strength assumed equal to the nominal design strengths.

The unit load calculated for Mechanism 1, which refers to the intact structure, confirms that the as-built structure had adequate flexural strength for code-prescribed gravity loads.

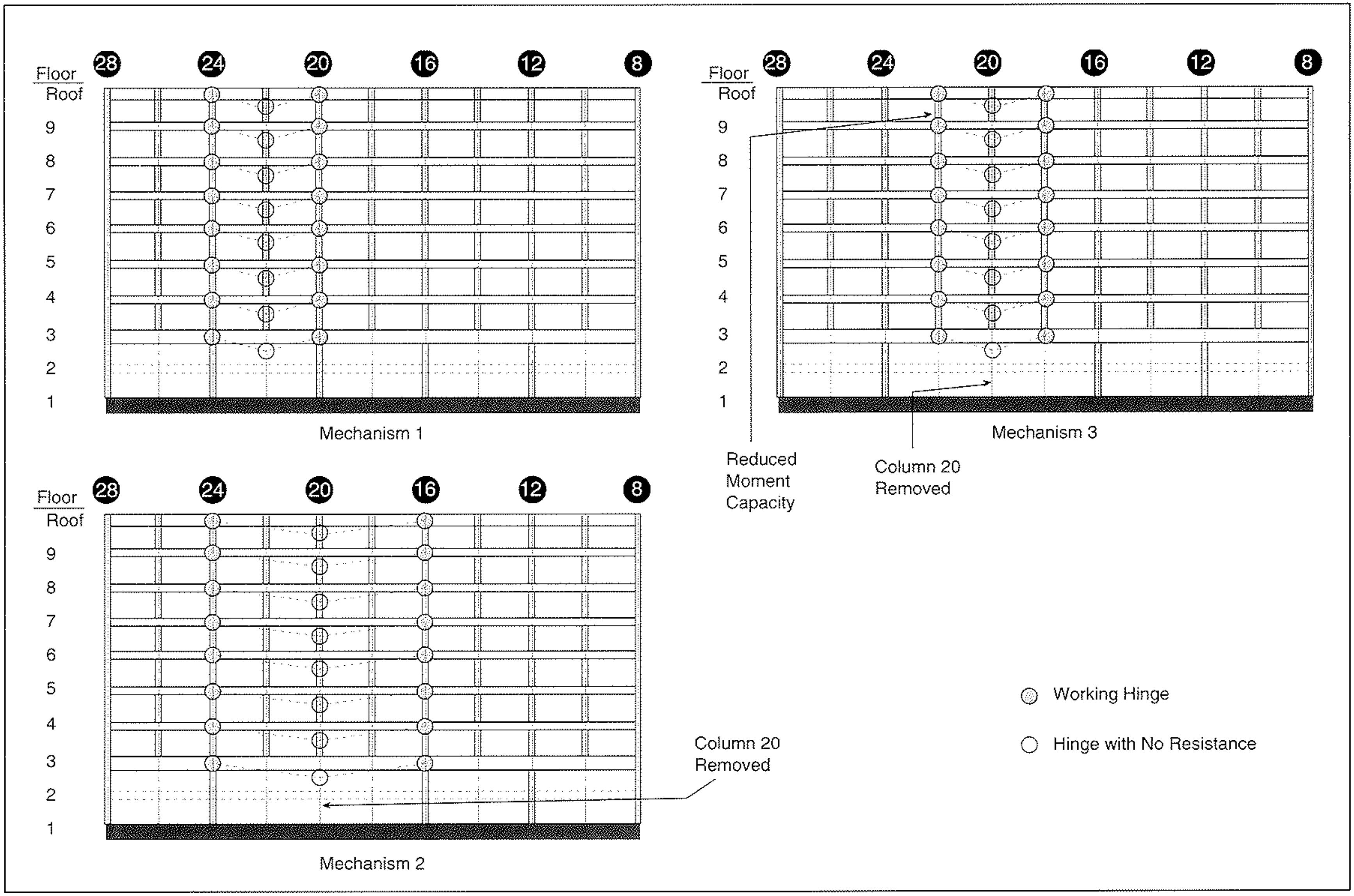


Figure 3-30 *Collapse Mechanisms 1, 2, and 3 (north elevation of nine-story portion of MurrahBuilding).*

Two-Dimensional Limit Analysis for As-Built Structure – Nominal Strength

Assumed Tributary Width (feet)	20
Supporting Column Width (inches)	16 (Based on minimum)
Span – Centerline to Centerline (feet)	40
Span – Face to Face of Negative Moment Hinges (feet)	38.67
Virtual Displacement at Positive Moment Hinge	1
Virtual Rotation (1 / feet)	0.0517

Internal Virtual Work

Floor	Moment – (thousands of pound-feet)	Moment + (thousands of pound-feet)	Internal Virtual Work
Roof	260	0	13
9	540	430	50
8	540	430	50
7	540	430	50
6	540	430	50
5	540	430	50
4	540	430	50
3	4,530	4,150	449
		Total Internal Virtual Work for One Half	763
		Total Internal Virtual Work	1,527

External Virtual Work

Tributary Width (feet)	20
Length (feet)	38.67

Floor	Relative Load	Total Load	External Virtual Work
Roof	1	773	387
9	1	773	387
8	1	773	387
7	1	773	387
6	1	773	387
5	1	773	387
4	1	773	387
3	1	773	387
		Total External Virtual Work	3,093

Unit Load at Limit (pounds per square foot)	**490**
Strength Reduction (phi = 0.9)	440

Table 3-8 Calculated unit strengths — Mechanism 1 (design yield stress = 60,000 pounds per square inch).

Two-Dimensional Limit Analysis, Column G20 Removed at First Floor – Nominal Strength

Assumed Tributary Width (feet)	20
Supporting Column Width (inches)	16 (Based on minimum)
Span – Centerline to Centerline (feet)	80
Span – Face to Face of Negative Moment Hinges (feet)	78.67
Virtual Displacement at Positive Moment Hinge	1
Virtual Rotation (1 / feet)	0.0254

Internal Virtual Work

Floor	Moment – (thousands of pound-feet)	Moment + (thousands of pound-feet)	Internal Virtual Work
Roof	260	0	7
9	540	0	14
8	540	0	14
7	540	0	14
6	540	0	14
5	540	0	14
4	540	0	14
3	4,530	0	115
		Total Internal Virtual Work for One Half	204
		Total Internal Virtual Work	408

External Virtual Work

Tributary Width (feet)	20
Length (feet)	78.67

Floor	Relative Load	Total Load	External Virtual Work
Roof	1	1573	787
9	1	1573	787
8	1	1573	787
7	1	1573	787
6	1	1573	787
5	1	1573	787
4	1	1573	787
3	1	1573	787
		Total External Virtual Work	6,293

Unit Load at Limit (pounds per square foot)	**60**
Strength Reduction (phi = 0.9)	50

Table 3-9 *Calculated unit strengths — Mechanism 2 (design yield stress = 60,000 pounds per square inch).*

Two-Dimensional Limit Analysis, Partial Mechanical, Column G20 Removed – Nominal Strength

Assumed Tributary Width (feet)	20
Supporting Column Width (inches)	16 (Based on minimum)
Span – Centerline to Centerline (feet)	40
Span – Face to Face of Negative Moment Hinges (feet)	38.67
Virtual Displacement at Positive Moment Hinge	1
Virtual Rotation (1 / feet)	0.0517

Internal Virtual Work

Floor	Moment – (thousands of pound-feet)	Moment + (thousands of pound-feet)	Internal Virtual Work
Roof	260	0	13
9	250	0	13
8	250	0	13
7	250	0	13
6	250	0	13
5	250	0	13
4	250	0	13
3	1,310	0	68
		Total Internal Virtual Work for One Half	159
		Total Internal Virtual Work	318

External Virtual Work

Tributary Width (feet)	20
Length (feet)	38.67

Floor	Relative Load	Total Load	External Virtual Work
Roof	1	773	387
9	1	773	387
8	1	773	387
7	1	773	387
6	1	773	387
5	1	773	387
4	1	773	387
3	1	773	387
		Total External Virtual Work	3,093

Unit Load at Limit (pounds per square foot)	**100**
Strength Reduction (phi = 0.9)	90

Table 3-10 Calculated unit strengths — Mechanism 3 (design yield stress = 60,000 pounds per square inch).

Two-Dimensional Limit Analysis for As-Built Structure – Nominal Strength

Assumed Tributary Width (feet)	20
Supporting Column Width (inches)	16 (Based on minimum)
Span – Centerline to Centerline (feet)	40
Span – Face to Face of Negative Moment Hinges (feet)	38.67
Virtual Displacement at Positive Moment Hinge	1
Virtual Rotation (1 / feet)	0.0517

Internal Virtual Work

Floor	Moment – (thousands of pound-feet)	Moment + (thousands of pound-feet)	Internal Virtual Work
Roof	300	0	16
9	630	490	58
8	630	490	58
7	630	490	58
6	630	490	58
5	630	490	58
4	630	490	58
3	4,530	4,150	449
		Total Internal Virtual Work for One Half	812
		Total Internal Virtual Work	1,624

External Virtual Work

Tributary Width (feet)	20
Length (feet)	38.67

Floor	Relative Load	Total Load	External Virtual Work
Roof	1	773	387
9	1	773	387
8	1	773	387
7	1	773	387
6	1	773	387
5	1	773	387
4	1	773	387
3	1	773	387
		Total External Virtual Work	3,093

Unit Load at Limit (pounds per square foot)	**530**
Strength Reduction (phi = 0.9)	480

Table 3-11 *Calculated unit strengths — Mechanism 1 (measured yield stress = 70,000 pounds per square inch).*

Two-Dimensional Limit Analysis, Column G20 Removed at First Floor – Nominal Strength

Assumed Tributary Width (feet)	20
Supporting Column Width (inches)	16 (Based on minimum)
Span – Centerline to Centerline (feet)	80
Span – Face to Face of Negative Moment Hinges (feet)	78.67
Virtual Displacement at Positive Moment Hinge	1
Virtual Rotation (1 / feet)	0.0254

Internal Virtual Work

Floor	Moment – (thousands of pound-feet)	Moment + (thousands of pound-feet)	Internal Virtual Work
Roof	300	0	8
9	630	0	16
8	630	0	16
7	630	0	16
6	630	0	16
5	630	0	16
4	630	0	16
3	4,530	0	115
		Total Internal Virtual Work for One Half	219
		Total Internal Virtual Work	438

External Virtual Work

Tributary Width (feet)	20
Length (feet)	78.67

Floor	Relative Load	Total Load	External Virtual Work
Roof	1	1573	787
9	1	1573	787
8	1	1573	787
7	1	1573	787
6	1	1573	787
5	1	1573	787
4	1	1573	787
3	1	1573	787
		Total External Virtual Work	6,293

Unit Load at Limit (pounds per square foot)	**70**
Strength Reduction (phi = 0.9)	60

Table 3-12 *Calculated unit strengths — Mechanism 2 (measured yield stress = 70,000 pounds per square inch).*

Two-Dimensional Limit Analysis, Partial Mechanical, Column G20 Removed – Nominal Strength

Assumed Tributary Width (feet)	20
Supporting Column Width (inches)	16 (Based on minimum)
Span – Centerline to Centerline (feet)	40
Span – Face to Face of Negative Moment Hinges (feet)	38.67
Virtual Displacement at Positive Moment Hinge	1
Virtual Rotation (1 / feet)	0.0517

Internal Virtual Work

Floor	Moment – (thousands of pound-feet)	Moment + (thousands of pound-feet)	Internal Virtual Work
Roof	300	0	16
9	290	0	15
8	290	0	15
7	290	0	15
6	290	0	15
5	290	0	15
4	290	0	15
3	1,310	0	68
		Total Internal Virtual Work for One Half	173
		Total Internal Virtual Work	347

External Virtual Work

Tributary Width (feet)	20
Length (feet)	38.67

Floor	Relative Load	Total Load	External Virtual Work
Roof	1	773	387
9	1	773	387
8	1	773	387
7	1	773	387
6	1	773	387
5	1	773	387
4	1	773	387
3	1	773	387
		Total External Virtual Work	3,093

Unit Load at Limit (pounds per square foot)	**110**
Strength Reduction (phi = 0.9)	100

Table 3-13 *Calculated unit strengths — Mechanism 3 (measured yield stress = 70,000 pounds per square inch).*

Comparison of the results for Mechanisms 2 and 3 indicates that the latter is not likely to govern. The nominal unit strength calculated for Mechanism 2 is 60 pounds per square foot, less than 15 percent of the as-built (flexural) strength of the structure. Considering that the unit load of the building is likely to be in the range 150 to 200 pounds per square foot, removal of Column G20 is concluded to be sufficient cause for failure. The strength of 70 pounds per square foot calculated for this mechanism with the higher measured yield stress of 70,000 pounds per square inch (#7 and #8 reinforcing bars) may be increased another 40 percent by credible increases in strength, amount of reinforcement, and effects of three-dimensional response, but to increase the calculated strength by over a factor of 2 to a level comparable to the self-weight of the structure is not plausible.

Strictly, the results for Mechanisms 1 and 2 are both hypothetical. For Mechanism 1, there would be problems associated with shear. The calculated load represents a measure of the strength of Column Line G if its strength would be limited by flexure of the girders. Mechanism 2 assumes that the deformations will be tolerable after failure of the positive-moment hinges at Column Line 20. The decisive conclusion from the comparison between the result of Mechanism 2 and the dead load is that Column Line G cannot sustain its tributary weight if any one of the interior columns is removed.

3.3.5 Analysis of Results

From the calculations summarized above, it can be concluded that even a "static" removal of Column G20 at the first floor would create sufficient reason for structural collapse of Column Line G between Column Lines 16 and 24. The events were neither static nor describable by a single variable in one dimension. The failure may be explained as a result of complex interaction of many events in many directions, but Mechanism 2 provides a fundamental and simple reason for the failure. The structure is not stable without Column G20.

Column G20 most likely failed in a mode indicated by the preceding analysis. It is unlikely that there was time for a flexural failure. The column would have been engulfed before the required movement could occur, dissipating most of the applied pressure. A strong possibility is that the blast "scoured" part of the shell immediately. The core, with its light transverse reinforcement and heavy longitudinal reinforcement, was brittle. It shattered (in shear) partially before being engulfed. (Shear cracks could have occurred with the shell intact.) The column reaction related to self-weight demolished whatever was left intact of the column section.

Calculations in Section 3.2 show that Columns G16 and G24 at the first floor were also vulnerable to the same failure sequence. However, they could have been destroyed by being pulled down by the connections of the falling spandrels. The remaining stub of the transfer girder framing into Column G12 suggests that the discontinuity of the top reinforcement would have isolated the failure had the columns not failed first.

For reasons of convenience in laying reinforcing bars, the positive-moment reinforcing bars appear to have been extended through supports for the spandrel girders at the fourth through the ninth floors. That condition helped increase the load calculated for Mechanism 1. Had this detail also been used across Column Lines 12, 16, 20, and 24 for all girders, the load calculated for Mechanism 2 would also have increased to 120 pounds per square foot (nominal material strength), which is within range of the self-weight required for Ordinary Moment Frames.

Spiral (helical) reinforcement prescribed for Special Moment Frames by Chapter 21 of the ACI's *Building Code Requirements for Reinforced Concrete* (ACI 318-89, Reference 3), but not required for Ordinary Moment Frames by ACI 318-71 (Reference 2), would have resisted the shattering of the column and would have maintained axial-load strength after scouring of the shell. Use of spiral reinforcement, especially in cases with discontinued column arrangements as in the Murrah Building and close bursts, may be an important ingredient for blast resistance.

3.4 SUMMARY

From the analyses described in this section, the blast was equivalent to the detonation of 4,000 pounds of TNT. The blast caused the removal of Column G20 by brisance as well as the shear failure of Columns G16 and G24. With this loss of three intermediate principal columns, the transfer girder supporting the upper portion of the building on the west side collapsed. Most of the devastation was due to this progressive collapse rather than the direct effects of the explosion. Limit analyses of Column Line G (Figure 3-30) indicate that the frame does not have the capacity to resist its self-weight if any one of the first-story columns on Column Line G is lost.

Use of spiral reinforcement in the first-story columns and changes in detailing requirements for the reinforcing bars in new construction are good candidates for avoiding general damage from close bursts. ("Detailing" is the process of selecting and designating on drawings the amounts, lengths, bends, and locations of steel reinforcement in reinforced concrete.) Such reinforcement of columns can contribute to preventing disintegration of columns close to the blast. If the columns stay in place, they can help prevent progressive collapse even though badly damaged.

Continuous reinforcement in transfer girders can provide continuity that will permit spanning the gap if one column is removed. In the Murrah Building, column shear reinforcement in the form of closed hoops or spirals could have prevented failure of Columns G16 and G24. In combination with continuity reinforcement, column shear reinforcement would have greatly reduced progressive collapse.

4 Findings and Conclusions

4.1 Relevant Factors

Based on review of plans, shop drawings, specifications, and construction records, it is concluded that the Murrah Building was designed as an ordinary reinforced-concrete-frame structure in accordance with ACI 318-71 (Reference 2). Records indicate that the building was extremely well detailed.

The structural design was found to have included all of the factors required by the governing building code at the time of construction and to have been extremely well executed. When this building was designed, there was no requirement to consider earthquake, blast, or other extreme loadings in Oklahoma City.

According to the observations made of the crater and other damage, the blast that damaged the building had a yield equivalent to approximately 4,000 pounds of TNT. This extremely large explosion was centered approximately 15.6 feet from Column G20 (see Figure 3-3). The blast caused a crater approximately 28 feet in diameter.

The effect of the blast on Column G20 abruptly removed it by brisance (a shattering effect). Loss of this column would remove support for the transfer girder on the third floor between Columns G16 and G24. Analysis of mechanisms that could result when Column G20 was removed shows that an Ordinary Moment Frame would be unable to support the structure above the third floor.

In addition to destroying Column G20, force from the blast would cause Columns G16 and G24 to be loaded in such a way that yield or near-yield moments would be produced over their lengths from their bases to the third floor. Corresponding shear stresses would exceed the calculated shear capacity of each column. Consequently, calculations indicate that Columns G16 and G24 failed in shear. Loss of these two columns would leave the transfer girder unsupported from the east wall of the building to Column Line G12. Calculations indicate that an Ordinary Moment Frame could not support itself with three columns in Column Line G missing.

It is noted that the loss of three columns and portions of some floors by direct effects of the blast accounted for only a small portion of the damage. Most of the damage was caused by progressive collapse following loss of the columns.

4.2 Possible Mechanisms for Reducing Loss

Ordinary Moment Frames have limited reserves for dissipating energy from extreme loading such as earthquake and blast. However, Special Moment Frames and Dual Systems with Special Moment Frames, as defined in the 1994 edition of *NEHRP (National Earthquake Hazards Reduction Program) Recommended Provisions for Seismic Regulations for New Buildings* (Reference 7), provide structural systems with much higher ability to dissipate energy. It is noted that the NEHRP recommendations for design of Special Moment Frames and Dual Systems were first available in 1985, approximately 10 years after the building was constructed.

If the more recently developed detailing for Special Moment Frames had been present at the time of the blast, Columns G16 and G24 would have had enough shear resistance to develop a mechanism without failure. Consequently, it is likely that G16 and G24 would not have failed abruptly due to the blast loading if Special Moment Frame detailing had been used.

Due to its close proximity to the very large explosive device, Column G20 would be likely to have been destroyed by brisance even if it was detailed as a Special Moment Frame. However, the heavy confinement reinforcement that would have been present would have increased the chances of survival for Column G20.

If Special Moment Frame detailing had been used, the following results could have been expected:

1. If Column G20 survived the blast, loss of structure would have been limited to those floor slabs destroyed by air blast. This would reduce the loss of floors by as much as 85 percent.

2. If Column G20 was removed by the blast, Mechanism 2 as described in Section 3.3 would develop. Normal detailing for Special Moment Frame design would provide reinforcement in the transfer girder at the third floor that would greatly increase the possibility that the slabs above would not collapse. Consequently, destruction could be limited to only those areas described in Section 3.2 as being removed by air blast. Although use of a Special Moment Frame would not completely eliminate loss of portions of the building, it is estimated that losses would be reduced by as much as 80 percent.

3. If Column G20 was removed by the blast and Mechanism 2 developed but was not capable of supporting the spans between Columns G16 and G24, loss of the structure would be limited to those panels destroyed by air blast and those panels located between Column Lines F to G and Column Lines 16 to 24. Resulting loss of floor space to either air blast or collapse would be reduced by more than 50 percent.

5 Review of Strategies for Mitigation

In 1991, the ASCE created a task committee to produce a report describing the state of the practice in structural design for physical security. This document is summarized in the Proceedings of the 1996 ASCE Structures Congress and is expected to be publicly available by the end of 1996. The separate chapters of the report are coordinated to collectively describe the procedures by which structural design for physical security is currently accomplished. The document provides both methods guidance and references for structural engineers challenged with a physical security problem. The report contains eight chapters that parallel the steps currently practiced in structural design for physical security.

The first step is the determination of a rational threat for the design. This step involves consideration of the assets that are to be protected at a particular facility. Next, the level of protection is determined. Then, the likely aggressors and tactics to be employed are developed based on the public profile of the particular installation. The chapter of the report that discusses this step contains an algorithm through which a quantified level of terrorist attack is determined for a particular facility.

The next step in the process is the calculation of the engineering parameters of the structural loading associated with the quantified threat. For external explosions, defense information on peak pressure, duration, and wave form are the parameters of interest. These are typically related to the quantity and type of explosive, the range from the bomb to the target, and the angle of incidence for the particular structural element.

A structural system is then chosen to carry these loadings. An important consideration in this choice is the provision for ductile failure modes and redundant load paths so that the structure does not progressively collapse when overloaded. Special Moment Frame systems, Dual Systems, and "compartmentalized" construction are particularly good candidates as discussed in Section 6.

Next, the individual elements of these systems are sized for the dynamic loadings of the design bomb threat. Design is typically and effectively accomplished with single degree of freedom models. The analysis is usually performed with unfactored loads and resistances and provides for inelastic response and damage to the structural element.

Special Moment Frames and Dual Systems are described in the most recent update of *NEHRP (National Earthquake Hazards Reduction Program) Recommended Provisions for Seismic Regulations for New Buildings* (Reference 7). Executive Order (EO) 12699 (Reference 26) requires that new Federal buildings be designed to meet the seismic requirements recommended in Reference 7. Consequently, new Federal buildings in areas of high seismicity may already provide suitable ductility for blast resistance at no additional cost by satisfying seismic design requirements.

Occasionally, the function of the facility requires that secure windows be provided. This process first involves the selection of blast-resistant glazing consistent with the overpressure wave form of the design threat. It is important to provide a substantial structural frame to restrain the blast-resistant lite and to adequately anchor this frame to the overall structural system. For many facilities, a lower level of threat is used for these window systems than for the overall structural system itself.

Doors on secure facilities are another special consideration. They are first designed much like window systems. However, they also must be easily operable. For some facilities, depending on their function, the doors must also provide a level of resistance to forced entry.

Openings for the utility systems of blast-resistant structures are another special detail that must be considered in design. These openings must first resist the blast loading determined for the overall structural system. They have a unique constraint in that they must provide a mechanical or electrical function as well, such as passing a specified volume of air in normal operations.

The rehabilitation/retrofitting of existing facilities represents a particularly demanding task. In general, the same procedures practiced in the design of new facilities apply. However, the available solutions are necessarily more limited — for example, siting the facility at a significant range from the design threat to reduce the blast loading. Nonetheless, practical solutions have been provided in various cases. Requirements for seismic rehabilitation/retrofitting of existing Federal buildings are set forth in EO 12941 (Reference 25) and its associated seismic standards (Reference 27). EO 12941 is similar in objective to EO 12699 but specifically addresses existing rather than new buildings.

Analysis of the Murrah Building shows that it would have been impossible to design the building to remain standing with one of its critical columns destroyed by the blast through the use of brute strength alone. However, if the additional amounts and locations of reinforcing steel called for in a Special (as opposed to a an Ordinary) Moment Frame had been used, the Murrah Building would have had enough toughness and ductility that about half of the damage would have been prevented. That is, even though the individual columns and slabs would not have had enough strength to avoid being cracked, the reinforcing steel would have held many of the building elements in place, keeping large portions of the building erect (at least sufficiently erect to allow the occupants to escape after the blast).

Another loss-reduction technique is to prevent a bad situation from getting worse: to prevent progressive collapse. Redundancy is a key design feature for the prevention of progressive collapse. There should be no single critical element whose failure would start a chain reaction of successive failures that would take down a building. Each critical element should have one or more redundant counterparts that can take over the critical load in case the first should fail.

6 Recommendations

6.1 New Buildings

From visual inspection and analysis of the damage that occurred in the Murrah Building as a result of a blast caused by a truck bomb, it is concluded that progressive collapse extended the damage beyond that caused directly by the bomb. The type of damage that occurred and the resulting collapse of nearly half the building is in line with what would be expected for buildings of the type and detailing available in the mid 1970's, when the Murrah Building was designed and built, and for the large bomb that was detonated.

Since the Murrah Building was designed and constructed, FEMA, through the Building Seismic Safety Council of the National Institute of Building Sciences, has developed recommended structural design procedures for earthquake resistance in new buildings (Reference 7). Some of the structural systems detailed in these procedures provide significantly more toughness and ductility than buildings constructed in the 1970's. In addition, the Department of Housing and Urban Development (HUD) has sponsored research to determine ways of reducing the effects of progressive collapse in the event of unanticipated catastrophic loadings such as those caused by a bomb. Selected references from HUD-sponsored research may be found in Section 7, along with references to studies by the National Bureau of Standards (now known as the National Institute of Standards and Technology), the Department of Defense, and others.

From this information, it is possible to identify types of structural systems that would provide significant increases in toughness to structures subjected to catastrophic loading from events such as major earthquakes and blasts. One of these systems is compartmentalized construction. Two additional types of systems, used in areas of high seismicity, are Special Moment Frame (as opposed to Ordinary Moment Frame) construction and Dual Systems with Special Moment Frames (herein referred to as Dual Systems).

The compartmentalized type of construction has been proven to provide significant structural integrity and resistance to progressive collapse as shown in Figure 6-1. However, the small and inflexible spaces created by this type of construction are not well-suited to office buildings. For this reason, compartmentalized construction could not often be used in Federal buildings.

Special Moment Frames and Dual Systems are frequently used in areas of high seismic activity. Using reinforcement, connection, and other details originally developed to provide toughness and ductility under seismic loads will provide similar toughness and ductility in the face of blast. In areas of low seismic risk, incorporating the seismic detailing required for regions of high seismic risk can provide blast protection. The engineering analysis performed on the Murrah Building suggests that the higher lateral force levels required for seismic design in regions of high seismic risk are not required for blast mitigation in regions of low seismic activity; only the detailing requirements need be followed. Special Moment Frames and Dual Systems can provide very large open spaces. Consequently, they are suitable for construction of Federal office buildings.

Compartmentalized construction, Special Moment Frames, and Dual Systems provide the mass and toughness necessary to reduce the effects of extreme overloads on buildings. Consequently, it is recommended that these structural systems be considered where a significant risk of seismic and/or blast damage exists.

6.2 Existing Buildings

Generally, it is more difficult and expensive to retroactively add features to a building than it is to include them in the original design. Blast-resistant features are no exception. Site design is extremely difficult to change. The Murrah Building, for example, could not have been moved back a dozen yards from the sidewalk and streetside parking. Limiting access to vehicle bombs for existing buildings typically requires closing all or part of adjacent streets, a solution that, while possibly cost-effective, is not often practical for traffic circulation. Therefore, modifications to the building itself should be considered.

It is not possible to prevent all damage in the immediate area where the blast occurs. Not only can the exact location of a bomb never be pinpointed in advance, but hardening building parts such as walls, windows, and columns to resist the extreme impact is not feasible. However, steps can be taken to try to minimize the impact outside of the immediate blast area. Additional structural walls can be added to compartmentalize the building. Supplemental supporting frames can be added where more redundancy is needed. Column jacketing (encasing an existing column in steel and concrete) can be used to toughen critical columns. Many of the techniques used to upgrade the seismic resistance of buildings also improve a building's ability to resist the extreme loads of a blast and reduce the likelihood of progressive collapse following an explosion.

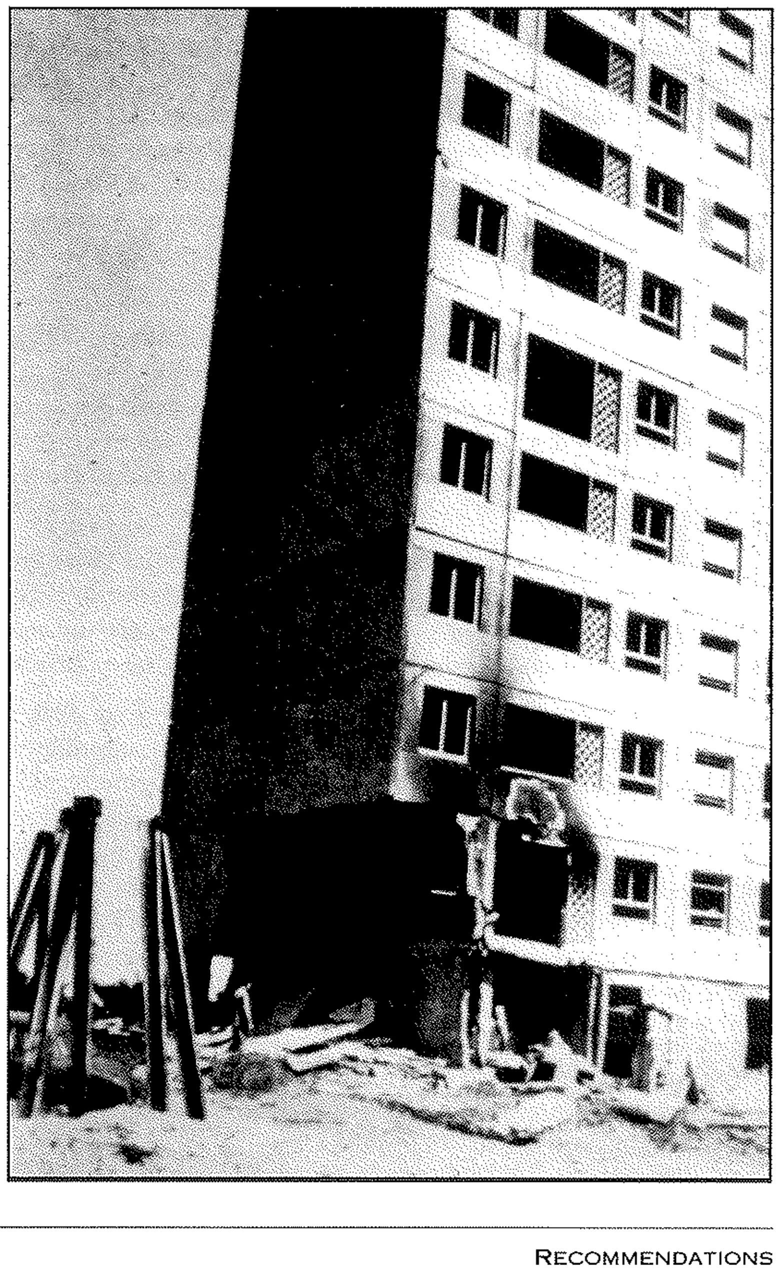


Figure 6-1
Demonstration of structural integrity of compartmentalized construction.

6.3 Economic Considerations

Investigations to determine the cost of using Special Moment Frames rather than Ordinary Moment Frames were conducted by the Building Seismic Safety Council. These investigations, along with more recent changes in designs, suggest that for new construction the average increase in cost is in the range of 1 to 2 percent of the total construction cost of the building. This additional cost can be expected to be within the normal differences between high and low construction bids on a Federal building. From these observations, it appears desirable and economically feasible to use compartmentalized construction, Special Moment Frame designs, or Dual Systems for Federal buildings where a significant risk of seismic and/or bomb damage exists.

When the benefits and costs associated with either seismic- or blast-resistant designs are being analyzed, the additional benefits derived from providing tangential blast resistance, in the case of seismic designs, and tangential seismic resistance, in the case of blast designs, should be considered. The inclusion of such additional benefits may provide economic justification for designs that offer greater toughness.

7 References

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Appendix A

BPAT Members

American Society of Civil Engineers (ASCE)

W. Gene Corley, Ph.D., P.E. – Vice President (BPAT Principal Investigator)
Construction Technology Laboratories, Inc.
Dr. Corley is a senior structural engineer with extensive experience in failure investigations, building codes, and reinforced concrete. He serves on several ASCE technical committees and on the ASCE Executive Committee of the Technical Council on Forensic Engineering.

Mete A. Sozen, Ph.D. – Kettlehut Distinguished Professor
Purdue University
Dr. Sozen is a senior structural engineer with over 35 years of experience in concrete and seismic design. He has served on several ASCE technical committees, including Limit Analysis, Design of Reinforced Columns, and Connections.

Charles H. Thornton, Ph.D., P.E. – Principal
Thornton - Tomasetti / Engineers
Dr. Thornton is a senior structural engineer with extensive experience in the investigation of collapse and structural failures.

Paul F. Mlakar, Ph.D., P.E. – Research Structural Engineer/
Chief, Concrete Technology Division
U.S. Army Corps of Engineers
Dr. Mlakar is a senior structural engineer experienced in blast-resistant design. He Chairs the ASCE Committee on Shock and Vibratory Effects and leads the Task Committee that is producing the state of practice report on Structural Design for Physical Security.

Federal Emergency Management Agency (FEMA)

Clifford Oliver – Senior Engineer / Project Officer and Team Leader
Mr. Oliver is a senior engineer with extensive experience in hazard-resistant construction and retrofit construction.

Paul Hoofnagle – Senior Engineer / Project Manager
Greenhorne & O'Mara, Inc. (FEMA Support Contractor, EMW-94-C-4421)
Mr. Hoofnagle is a senior engineer with experience in structural evaluation and mitigation design for seismic hazards.

U.S. Army Corps of Engineers (USACE)

Stanley C. Woodson, Ph.D., P.E. – Research Structural Engineer
Dr. Woodson is a senior structural engineer specializing in blast-resistant design.

National Institute of Standards and Technology (NIST)

Richard D. Marshall, Ph.D. – Research Structural Engineer
Dr. Marshall is a senior structural engineer with expertise in wind and blast loads.

General Services Administration (GSA)

David Kossover, P.E. – Structural Engineer
Mr. Kossover is a senior structural engineer with expertise in blast-resistant design.

Acknowledgments

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Ray Blakeney
Director of Operations
Oklahoma State Medical Examiner's Office

Lieutenant O. K. Cantrell
Deputy Sheriff / Range Master
Oklahoma County Sheriff's Department

Robert Cornforth, P.E.
Structural Engineer

James Goodbread
Fire Chief
Tinker Air Force Base

Captain Bob Heady
Technician / HDS
Commander, Bomb Disposal Unit
Oklahoma County Sheriff's Department

Robert Hill, P.E.
Structural Engineer
Brockett Associates

David Hammond, P.E.
Structural Engineer

Paul Ed Kirkpatrick, P.E.
Kirkpatrick Engineering Company

Don C. Liston
Engineering Print Shop Manager
City of Oklahoma City

Donald Lynch
Director, Oklahoma County
Department of Emergency Management

Ken Morris
NFIP State Coordinator
State of Oklahoma
Water Resources Board

Captain James D. Rouse
Chief of Operations / Bomb Technician
Oklahoma County Sheriff's Department

David L. Smith, Jr.
Visual Information Specialist
Federal Bureau of Investigation

Appendix B

SELECTED STRUCTURAL DETAILS AND DEADLOAD ANALYSIS

B-1 SELECTED DETAILS

The following discussion is based on the BPAT's review of the structural drawings for the Murrah Building.

TYPICAL FLOOR FRAMING

One-way concrete slabs (identified as "TS" on the structural drawings) spanned the east-west direction. These slabs were 6 inches thick and were reinforced with #4 bars 17 feet - 6 inches long at 18 inches on center and #4 bars 12 feet long at 18 inches on center. Therefore, the amount of reinforcing at the centerline of the span was # 4 bars at 9 inches bottom steel or 0.27 square inches of 60,000-pounds-per-square-inch-yield reinforcing bar per foot of width. The longer bottom bars extended 9 inches into the supporting T-beams at each end.

Top slab reinforcement centered over the column lines consisted of # 4 bars 10 feet long at 16 inches on center and # 4 bars 12 feet long at 16 inches on center. Therefore the amount of reinforcing was # 4 bars at 8 inches on center at the centerline of the support, or 0.30 square inches of reinforcing bar per foot of width.

The one-way slabs spanned 20 feet center to center in the east-west direction and are supported by T-beams that were 48 inches wide by 20 inches deep. Therefore, the face-of-support to face-of-support span for the one-way slabs was 16 feet. The longer of the top bars in the slab extended 4 feet - 3 inches into the slab span outside of the T-beam. As a result, there is no top steel in the slab for the middle 11-foot 6-inch portion of the 16-foot slab span.

The slabs contained temperature steel in the north-south direction consisting of #4 bars at 18 inches on center (0.135 square inch per foot). This steel was apparently placed directly on top of the bottom main east-west reinforcing bars. (Structural drawing 7-12 contains the note referring to temperature steel. Shop drawings that show how far the temperature steel extended into the spandrel beam at the north face of the building or how much lap occurred at splices were not available.)

The T-beams at the third floor and the typical floor framing north-south in the 35-foot span direction measured 48 inches wide by 20 inches deep. Reinforcing in these beams was as follows:

		Number - Size	Length
Column Lines 10, 12, 24, and 26			
T-Beam 6	Bottom (short)	2 - #7	22 feet - 6 inches
	Bottom at Column (long)	4 - #8	33 feet - 6 inches
	Top at Column Line G	4 - #10	12 feet - 0 inches
	Top at Column Line F	4 - #11	22 feet - 6 inches
	Stirrups	#4 at 9 inches	

		Number - Size	Length
Column Lines 16, 18, 20			
T-Beam 12	Bottom (short)	2 - #7	22 feet - 0 inches
	Bottom at Column (long)	4 - #8	33 feet - 6 inches
	Top at Column Line G	3 - #11	11 feet - 0 inches
	Top at Column Line F	3 - #10	22 feet - 6 inches
	Stirrups	#4 at 9 inches	
Column Lines 14, 22			
T-Beam 10	Bottom (short)	2 - #7	22 feet - 0 inches
	Bottom at Column (long)	4 - #8	33 feet - 6 inches
	Top at Column Line G	2 - #10	11 feet - 0 inches
	Top at Column Line F	4 - #11	23 feet - 6 inches
	Stirrups	#3 at 9 inches	

There was no continuous top steel across the span of the T-beams from Column Line G to Column Line F. The middle 12-foot 6-inch portion of the span was unreinforced at the top of the spandrel beam. Lack of continuous reinforcement was consistent with detailing required for Ordinary Moment Frame buildings. However, it is noted that a plane of weakness exists where main reinforcement is terminated.

At each typical floor there was an east-west spandrel beam along the north face that measured 18 inches wide by 35 inches deep. Reinforcing in these beams was as follows:

		Number - Size	Length
T-Beam 18	Bottom at Centerline	2 - #7	30 feet - 0 inches
	Bottom at Column	2 - #8	42 feet - 0 inche
	Side face	2 - #5	22 feet - 0 inches
	Top at Line	2 - #8	34 feet - 0 inches
	Stirrups	#6 at 17 inches	
	Stirrups	#4 at 17 inches	
T-Beam 19	Bottom at Centerline	(See T-beam 18)	
	Side face	2 - #5	22 feet - 0 inches
	Top at Column Line	2 - #9	24 feet - 0 inches
	Stirrups	#6 at 17 inches	
	Stirrups	#4 at 17 inches	
T-Beam 20	Bottom at Centerline	2 - #7 x	30 feet - 0 inches
	Bottom at Centerline	2 - #8 x	42 feet - 0 inches
	Side face	2 - #5 x	22 feet - 0 inches
	Top of Column Line	2 - #8 x	34 feet - 0 inches
	Stirrups	#6 at 17 inches	
	Stirrups	#4 at 17 inches	

Thus, these spandrel beams had at least two continuous #8 reinforcing bars at the top with a lap splice of 14 feet, resulting in a total cross-sectional area of tensile reinforcement for negative moment of two #8 bars (1.58 square inches) plus two #9 bars (2.0 square inches), or 3.58 square inches.

Column sizes along Column Line G from the third floor to the roof were typically 16 inches by 24 inches. Reinforcement consisted of four # 9 vertical bars at the lower levels and four # 8 vertical bars at the upper levels, with # 3 horizontal ties at 16 inches on center.

Third Floor Framing

In order to enhance the street level and open the building to allow access, transfer girders were located along Column Line G at the third floor. These girders spanned 40 feet, thus picking up the columns located at Column Lines 10, 14, 18, 22, and 26. Columns that were spaced 40 feet on center were therefore located at Column Lines 12, 16, 20, and 24. These columns measured 20 inches by 36 inches; the 36-inch dimension was in the north-south direction. Reinforcement consisted of 20 #11 vertical bars and #4 horizontal ties at 16 inches on center. The transfer girders at the third floor (with designations "3B-3" and "3B-4" on the structural drawings) measured 36 inches wide by 60 inches deep and contained heavy reinforcement. Reinforcing in these girders was as follows:

		Number - Size	Length
Between Column Lines 16 and 20			
Girder 3B-3	Bottom at Centerline	3 - #11	20 feet - 0 inches
		3 - #11	27 feet - 6 inches
		4 - #11	40 feet - 0 inches
	Top at Column Line	4 - #11	20 feet - 0 inches
		4 - #11	30 feet - 0 inches
		3 - #11	45 feet - 0 inches
	Stirrups	#6 at 6 inches	
Between Column Lines 20 and 24			
Girder 3B-4	Bottom at Centerline	(Same as 3B-3)	
	Top at Column Line	4 - #11	20 feet - 0 inches
		4 - #11	32 feet - 6 inches
		3 - #11	45 feet - 0 inches
	Stirrups	#6 at 6 inches	

The bottom steel configuration in the transfer girders was such that the longest bars (four #11's) were 40 feet long. Thus no lap occurred at the support columns at Column Lines G12, G16, G20, and G24.

The top steel configuration in these transfer girders was such that the three #11 continuous top bars left a 5-foot lap at the center of the 40-foot span at Column Lines 14, 18, 22, and 26. Negative moment caused by removal of Column G16 resulted in failure in 3B-2 at Column Line 14. Once the negative moment failure occurred at Column Line 14, the bottom reinforcement in 3B-2 was ripped out, which left the transfer girder stub observed between G14 and G12.

Second Floor Framing

The edge of the slab at the second floor was set in approximately 9 feet - 6 inches from Column Line G. Spandrel Beams 2B-33 and 2B-34 had cross sections 18 inches by 48 inches and spanned 40 feet. These spandrels picked up portions of the facade load, the third floor slab load, and the north end reaction of second floor Beams 2B-22 and 2B-17. The following reinforcement was provided:

		Number - Size	Length
Beam 2B-33	Bottom at Span Centerline	2 - #7	26 feet - 0 inches
		2 - #8	41 feet - 0 inches
	Top at Column Line	3 - #9	22 feet - 6 inches
	Top at Span Centerline	2 - #6	22 feet - 6 inches
	Stirrups	#5 at 22 inches	
Beam 2B-34	(Same as 2B-33)		

Thus, top steel at the centerline of the span consisted of two #6 bars with a lap splice of 2 feet - 6 inches and three #9 bars. These spandrel beams were supported by a concrete wall column 12 inches wide by about 10 feet long with 24 #7 vertical bars and #4 horizontal bars at 12 inches on center.

Roof Structure

The roof slabs (designated "RS-2" on the structural drawings) spanned, as on all other floors, in an east-west direction and were 6 inches thick, 17 feet - 6 inches long, and reinforced with # 4 bottom bars at 16 inches on center. The total cross-sectional area of tensile reinforcement at the centerline was 0.15 square inch per foot of slab width. The bottom bars extended 9 inches into the supporting beams (designated "RB" at each end on the structural drawings).

Top steel centered over the column lines consisted of # 4 bars 10 feet long at 24 inches on center and # 4 bars 12 feet - 6 inches long at 24 inches on center. Therefore, the amount of reinforcing was # 4 bars at 12 inches on center at the centerline of the support, or 0.20 square inch per foot of slab.

The reinforcing configuration and dimensions are the same as for the typical floors.

The roof beams (designated "RB" on the structural drawings) spanned 35 feet in the north-south direction and were 48 inches wide by 20 inches deep. Reinforcing in these beams was as follows:

		Number - Size	Length
Beam RB-4	Bottom short	2 - #8	20 feet - 0 inches
	Bottom at Column (long)	3 - #8	34 feet - 0 inches
	Top at Column Line G	4 - #8	10 feet - 6 inches
	Top at Column Line F	4 - #10	23 feet - 0 inches
		2 - #8	15 feet - 6 inches
	Stirrups	None	

There was no continuous steel across the span of the roof beams from Column Line G to Column Line F. The middle 12-foot 6-inch portion of the span was unreinforced at top of the roof beams.

The spandrel beam in the east west direction was 18 inches wide by 48 inches deep and contained the following reinforcing:

		Number - Size	Length
Beam RB-13	Bottom at Centerline Sideface	2 - #7	20 feet - 0 inches
	Top at Column Line	2 - #7	22 feet - 0 inches
	Stirrups	4 - #5 at 24 inches each end	

The roof spandrel beam reinforcing is different than the typical floor. There is no lap at the column line in the bottom steel, and there is continuous top reinforcing with a 2-foot lap at the centerline of the span. The lack of a lap in bottom steel creates a plane of weakness when unanticipated loads such as blast forces cause positive moment at columns.

Interior Columns

Interior Column F22 was 24 inches by 24 inches from the ground up to the third floor. Reinforcement consisted of 16 #11 vertical bars at the first floor, and 12 #11 vertical bars at the second floor with #4 horizontal ties at 16 inches on center. From the third floor to the roof, the column was 20 feet by 20 feet and contained the following reinforcing:

Floor	Vertical Bars (Number - Size)	Ties (Number and Spacing)
3 - 4	16 - #11	#4 at 16 inches on center
4 - 5	12 - #11	#4 at 16 inches on center
5 - 6	8 - #10	#3 at 16 inches on center
6 - 7	4 - #10	#3 at 16 inches on center
7 - 8	4 - #9	#3 at 16 inches on center
8 - 9	4 - #9	#3 at 16 inches on center
9 - Roof	4 - #8	#3 at 16 inches on center

Foundation

The foundation for the building is a drilled pier system with allowable soil bearing pressures for dead load plus 50 percent live load of 30,000 pounds per square foot at elevation 1194 to 1200 m.s.l., and 50,000 pounds per square foot at elevation 1175 to 1194 m.s.l. The lengths of drilled piers shown on drawings range from 20 feet to 30 feet.

Drilled piers supporting the building have large belled bottoms with diameters varying from 24 inches to 96 inches. Smaller shaft diameters vary from 18 inches to 48 inches.

Dowels

All columns are doweled into the top of the pier below. At the 20-inch by 36-inch columns, eight #10 dowels extend into a 48-inch-diameter drilled pier, and at the 12-inch by 120-inch wall column, four #7 dowels extend into each 24-inch-diameter drilled pier.

B-2 Dead Load Analysis

The Alfred P. Murrah Federal Building contained a roof, floors 9 through 2, and a ground floor. The eighth through fourth floors are typical.

For blast loading analyses of the various structural components, the dead load effects are required. Based on the structural size of all components in the general area bounded by Column Lines G and F and Column Lines 12 and 26, these loads have been calculated.

The average dead loads due to structural weight only are as follows:

Exterior Bay	Load (pounds per square foot)
Roof	203
Ninth Floor	152
Eighth - Fourth Floors	152
Third Floor	231
Second Floor	N/A

Interior Bay	Load (pounds per square foot)
Roof	117
Ninth Floor	117
Eighth - Second Floors	117

The tributary area for a typical exterior column above the third floor is 350 square feet per floor. The tributary area for a typical interior column above the first floor is 700 square feet per floor.

The total area supported by each of the Columns G16, G20, and G24 is 5600 square feet. Thus the axial force in these columns due to the dead load (self-weight) of the structures is 940,000 pounds. One half the weight of the column adds 10,000 pounds so that the axial force at midheight of these columns is 950,000 pounds.

The average superimposed dead load on the typical floors is as follows:

Interior Bay Areas		Range (pounds per square foot)
Partitions		0 - 20
Ceilings		1 - 2
MEP		2 - 4
Flooring		0 - 2
	Total	**3 - 28 pounds per square foot**

Exterior Bay Areas		Range (pounds per square foot)
Partition		0 - 20
Ceilings		1 - 2
MEP		2 - 4
Floorings		0 - 2
Exterior Facade		15 - 15
	Total	**18 - 43 pounds per square foot**

Since many aspects of the blast analysis cause more critical loading conditions for lower axial force in columns and lower dead loads on floors, the following range of loads from lower bound to upper bound is provided.

Lower Bound	G20	F20
Column Load	at third floor	at second floor
No Live Load		
Dead Load	950,000 pounds	37,000 pounds
Superimposed	81,000 pounds	20,000 pounds
Total	**1,031,000 pounds**	**957,000 pounds**

Mid Range	G20	F20
Column Load	at third floor	at second floor
1/2 Live Load		
Dead Load	950,000 pounds	737,000 pounds
(Superimposed)	137,000 pounds	98,000 pounds
1/2 Live Load	112,500 pounds	157,500 pounds
Total	**1,199,500 pounds**	**992,500 pounds**

Upper Bound	G20	F20
Column Load	at third floor	at second floor
Full Live Load		
Dead Load	950,000 pounds	737,000 pounds
(Superimposed)	193,500 pounds	176,400 pounds
Full Live Load	225,000 pounds	315,000 pounds
Total	**1,368,500 pounds**	**1,228,400 pounds**

The actual distribution of the dead load and live load between Column Lines G and F will not be based on a purely geometric tributary area calculation. With the two-span configuration of the north-south T-beams and roof beams, the floor load distribution will be 25 percent less for Column Line G and 25 percent higher for Column Line F.

As a result of this structural behavior, the lower bound for the axial force in Column G20 could be as low as 800,000 pounds.

Appendix C

PETROGRAPHIC REPORT

The following is a copy of the report prepared by Construction Technology Laboratories, Inc., concerning the tests performed on core samples taken from concrete from the Alfred P. Murrah Federal Building in Oklahoma City, Oklahoma.

Report to

GREENHORNE & O'MARA
9001 Edmonston Road
Greenbelt, MD 20770

PETROGRAPHIC EXAMINATION OF CORE SAMPLE EXTRACTED FROM CONCRETE FRAGMENT FROM ALFRED P. MURRAH FEDERAL BUILDING, OKLAHOMA CITY, OKLAHOMA

RONALD D. STURM

Submitted by

CONSTRUCTION TECHNOLOGY LABORATORIES, INC.
5420 Old Orchard Road
Skokie, IL 60077

April 23, 1996

CTL Structural/Architectural Engineering,
Testing and Materials Technology

5420 Old Orchard Road, Skokie, Illinois 60077-1030
847/ 965-7500 **800/ 522-2CTL** Fax: 847/ 965-6541

PETROGRAPHIC SERVICES REPORT

CTL Project No.: 200087 *Date:* April 22, 1996

Re: Petrographic Examination of Core Sample Extracted From Concrete Fragment From Alfred P. Murrah Federal Building, Oklahoma City, Oklahoma

One concrete core designated as "7B" (Fig. 1) was received from Dr. W. Gene Corley, Vice President, CTL. The core represents a portion of a larger concrete fragment (Fig. 2) reportedly taken from remains of the Alfred P. Murrah Federal Building in Oklahoma City. Petrographic examination of the core was requested by Dr. Corley to describe the constituents and general condition of the concrete.

FINDINGS

The following findings are based on the results of the petrographic examination, detailed in the attached data sheets.

1. The core consists of well-graded, crushed limestone coarse aggregate (3/4-in. top size) and siliceous, natural sand uniformly dispersed in a portland cement paste (Fig. 3). No fly ash or other mineral admixtures were observed in the cement paste. The water-cement ratio is interpreted at 0.45 to 0.50, based on observed paste properties, including estimated percentage of unhydrated portland cement clinker particles (UPC's) in the paste.

2. Air content in the core is estimated at 2.5 to 4.5%, attributed mainly to entrapped air. Small spherical voids, typical of entrained air, are also present in the concrete.

3. Carbonation of cement paste extends 0.5 to 0.7 in. (13 to 18 mm) from the wood-formed end of the core and up to 0.1 in. (2.5 mm) from the other end of the core (Fig. 4).

4. The condition and quality of the examined concrete are relatively good. Aside from a slightly higher than normal occurrence of small entrapped air voids, the concrete

CTL

appears to be fairly well consolidated and exhibits no evidence of detrimental internal paste-aggregate reactions. Cracking exhibited in the core appears to be limited to a few shallow microcracks likely caused by restrained drying shrinkage. Cursory examination of the large concrete fragment from which the core was extracted revealed some additional cracking not observed in the core.

METHODS OF TEST

Petrographic examination of the concrete core was performed in accordance with ASTM C 856-83 (reapproved 1988), "Standard Practice for Petrographic Examination of Hardened Concrete." A slice of concrete approximately 1 in. thick was cut longitudinally from the core and one of the resulting surfaces was lapped and examined using a stereomicroscope at magnifications up to 45X. Surfaces of freshly broken concrete were also studied with the stereomicroscope. A small rectangular block was cut from the top 1.0 in. of the core, placed on a glass microscope slide with epoxy resin, and reduced to a thickness of approximately 20 micrometers (0.0008 in.). The thin section was examined using a polarized-light microscope at magnifications up to 400X to determine aggregate and paste mineralogy and microstructure.

Ronald D. Sturm
Petrographer
Petrographic Services

RDS/nem

200087

Attachments

CTL

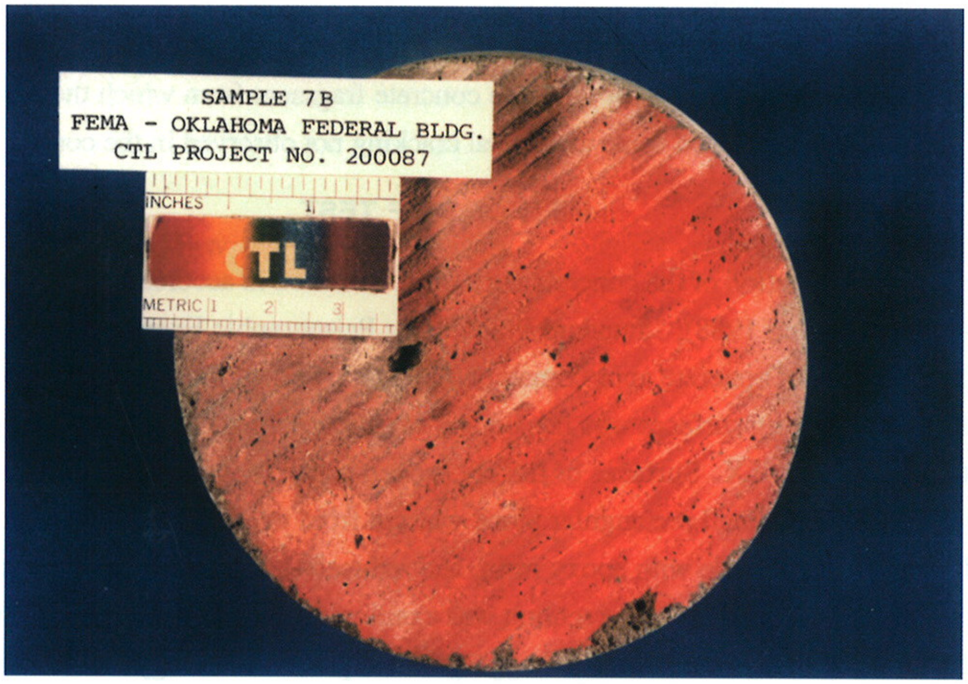

FIG. 1 END VIEW AND SIDE OF CORE 7B, AS RECEIVED FOR EXAMINATION.

CTL

FIG. 2 VIEW OF THE LARGE FRAGMENT OF CONCRETE (SAMPLE NO.7) AFTER TWO CORES WERE DRILLED. SAMPLE 7B, SELECTED FOR PETROGRAPHIC EXAMINATION, IS MARKED BY AN ARROW,. THE OTHER CORE WAS REMOVED AND FORWARDED TO CTL PHYSICAL TESTING FOR TESTING.

CTL

FIG. 3 LAPPED LONGITUDINAL CROSS SECTION OF CORE 7B SHOES AGGREGATE SIZE, SHAPE, GRADATION AND DISTRIBUTION OF THE CEMENT PASTE. NO MAJOR CRACKS ARE VISIBLE IN THE SAMPLE.

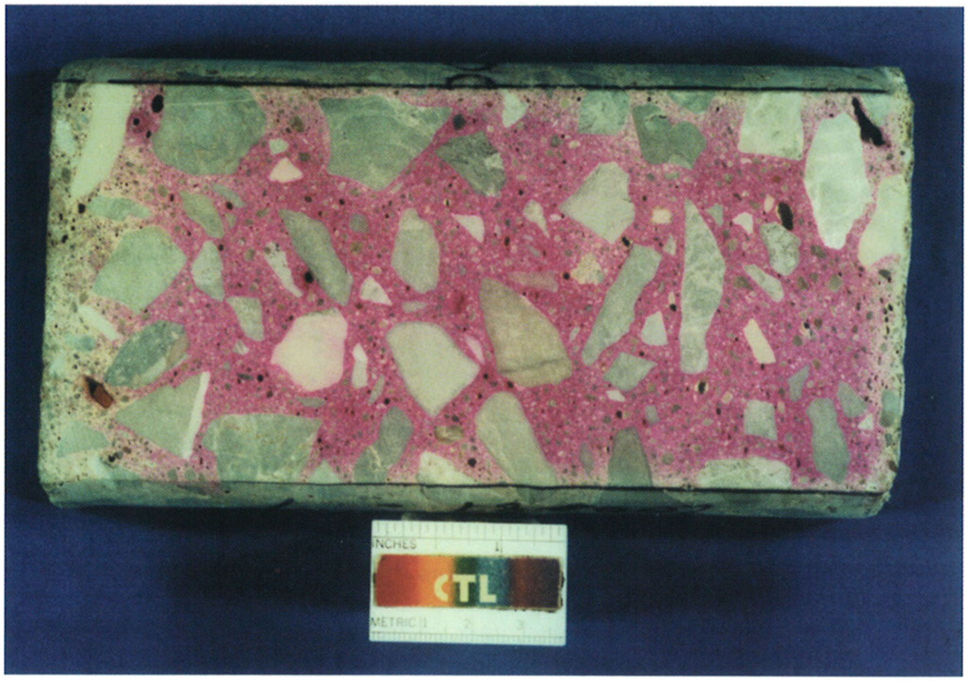

FIG. 4 OPPOSING, SAWCUT SIDE OF THE SLICE OF CONCRETE SHOWN IN FIG.3, AFTER THE SURFACE WAS TREATED WITH A pH INDICATOR SOLUTION. NONCARBONATED PASTE IS SHOWN STAINED A Magenta COLOR, WHILE LOWER pH CARBONATED PASTE REMAINS UNSTAINED.

CTL

PETROGRAPHIC EXAMINATION OF HARDENED CONCRETE, ASTM C 856

CTL PROJECT NO.: 200087

DATE: April 22, 1996

CLIENT: Not Stated

REPORTED PROBLEM: Condition Evaluation

STRUCTURE: Not Stated

EXAMINED BY: Ronald Sturm

LOCATION: Alfred P. Murrah Federal Building
Oklahoma City, Oklahoma

SAMPLE

Identification: Sample 7B.

Dimensions: Core diameter = 3.7 in. (94 mm); length = 6.5 in. (165 mm).

End Surfaces: One end is a flat formed surface with imprint of wooden forms. The other end is a slightly rough, flat, struck or lightly finished surface partially covered by insulation material.

Cracks, Joints, Large Voids: No joints, large cracks, or large voids are observed in the core. Several relatively small entrapped air voids, some of diameters up to 0.2 in. (5 mm), are randomly dispersed throughout the core.

Reinforcement: None present.

Unit Weight: 147.5 lb/cu ft, as received.

AGGREGATES (A)

Coarse (C): Crushed calcareous rock, mainly limestone and dolomitic limestone. The limestone contains variable, but relatively minor, amounts of chert, chalcedony, and detrital grains of quartz.

Fine (F): Natural sand containing grains of quartz, feldspar, quartzite, granite, chert, and other rocks and minerals.

Gradation & Top Size: Well graded to an observed top size of 0.75 in. (19 mm).

Shape & Distribution: Coarse aggregates are mainly angular and equant to elongate; fine aggregate particles are angular to subrounded and mainly equant; aggregate distribution is uniform.

PASTE

Color: Buff in the body of concrete; light to medium gray in the outer 0.1 in. at both ends of the core.

Hardness: Moderately hard; cement paste can be only slightly scratched using a steel dental probe.

Luster: Subvitreous to vitreous.

CTL

Depth of Carbonation: 0.5 to 0.7 in. (13 to 18 mm) from the wood-formed end of the core; up to 0.1 in. (2.5 mm) from the core end in contact with insulation material.

Air Content: Estimated 2.5 to 4.5%. The concrete appears to be air entrained, based on the presence of small, spherical voids (diameters <1 mm) in the cement paste. air-void distribution is somewhat nonuniform. Entrapped air content is estimated at 1.5 to 2.5% (included in estimated total air content, above).

Paste-Aggregate Bond: Moderately tight; surfaces of freshly fractured concrete pass mainly through calcareous coarse aggregates but mainly around harder, siliceous, sand grains.

Calcium Hydroxide*: Estimated 5 to 10%; CH crystals in the paste are relatively small, but some larger crystals (up to 50 µm) across) occur along periphery of some aggregates.

Unhydrated Portland Cement Clinker Particles (UPC's)*: Estimated 5 to 10%; distribution is somewhat nonuniform.

Pozzolans: None observed.

Secondary Deposits: Minor amounts of ettringite crystals ($3\ CaO \cdot A\ell_2O_3 \cdot 3\ CaSO_4 \cdot 32\ H_20$) occur as thin linings in some air voids. Minute crystals of calcium carbonate observed on the wood-formed end surface.

MICROCRACKING: A few microcracks extend to a depth of up to 0.5 in. (13 mm) from the wood-formed end of the core. A few random microcracks observed in coarse aggregates and the cement paste in the body of the core.

ESTIMATED WATER-CEMENT RATIO: 0.45 to 0.50, somewhat nonuniform.

MISCELLANEOUS: The remnants of insulation material attached to one end of the core consists mainly of vermiculite in a cementitious binder. No asbestos fibers were observed.

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*percent by volume of paste

Appendix D

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Figure	Source
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