Applicability of Seismic Design in Mitigating Progressive Collapse

by

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ABSTRACT

A Building Performance Assessment Team (BPAT) composed of American Society of Civil Engineers (ASCE) and Federal Government engineers investigated damage caused by the malevolent bombing of the Alfred P. Murrah Federal Building in Oklahoma City, Oklahoma. The purposes of the investigation were to review damage caused by the blast, determine the failure mechanism, and identify engineering strategies for reducing damage to new and existing buildings. Specifically, mechanisms for multi-hazard mitigation, including mitigation of earthquake effects, were considered. Among the strategies evaluated were use of seismic detailing for the structural concrete. This report describes results of the investigation, makes recommendations for design of buildings to be more blast-resistant, and discusses how these details might have changed results of damage in Oklahoma City. The paper also reviews requirements for structural integrity reinforcement after 1989, refers to a case study of blast-damage to the Murrah Federal Building in Oklahoma City, describes how earthquake detailing can reduce losses, and shows how the use of full capacity butt splices could have further reduced the casualties in that blast. Finally, it questions whether seismic detailing can protect buildings where brisance is the failure mode.

KEYWORDS: Blast damage, earthquakes, failure mechanism, multi-hazard mitigation, progressive collapse, seismic design.

1.0 INTRODUCTION - OKLAHOMA CITY BOMBING

In 1995, the Murrah Federal Office Building in Oklahoma City was heavily damaged by a terrorist bomb blast [1]. During the period, May 9 through 13, 1995, 3 weeks after the blast occurred on Wednesday, April 19, 1995, the BPAT visited the area around the Murrah Building in Oklahoma City. 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 recovered from a damaged building across the street from the Murrah Building. The team also conducted interviews of individuals involved in design, construction, and cleanup of damaged buildings. Physical inspection of the Murrah Building was limited to visual observation.

Upon completion of the site visit, data collected were analyzed and the most probable response of the building to the blast was determined. Using knowledge of building performance, an evaluation was made of the use of Special Moment Frame detailing to enhance the resistance of buildings to blast loading. Further details are provided in reference 1.

2.0 ALFRED P. MURRAH FEDERAL BUILDING

Design of the Murrah Building project, shown in figure 1, was done for the Design & Construction Division, Region 7, Fort Worth, Texas, of the GSA Public Buildings Service, Washington, DC. A 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 by The Ceco Company between December 1974 and May

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1975. A spot-check of the Ceco reinforcing bar shop drawings shows compliance and good correlation with the structural contract documents. Construction was completed in 20 months, between late 1974 and early 1976.

The Murrah Building project included a nine-story office building, shown in figure 1 (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.

Focus of this report is performance of the nine-story portion of the Murrah Building and on use of seismic detailing to provide improved resistance to bombing. The nine-story portion of the building sustained significant damage and progressive collapse as a result of the April 19, 1995 bombing as shown in figure 2. Remaining portions of the nine-story and one-story buildings were demolished shortly after completion of the investigation. The parking structure sustained little damage and was not immediately demolished.

The nine-story frame of the Murrah Building was an Ordinary Moment Frame of reinforced concrete supported on columns. Overall plan dimensions were approximately 61 m (220 ft) in the east-west direction and approximately 30.5 m (100 ft) in the north-south direction.

In plan, the structure for the nine-story portion consisted of ten 6.1 m (20 ft) bays in the east-west direction and two 10.7 m (35 ft) 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. Exposed reinforced concrete with a vertical-board-formed finish served as the exterior architectural treatment. Four large, prominent, vertical circular tube columns, one at each of the four corners of the building, acted as air intake/exhaust shaft for the ventilation system, as seen in figure 1. An elevator shaft and stair wells were located in the south central portion of the building.

Details of Murrah Building—Structural drawings that were obtained confirmed 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 neither the governing building code nor the owner required consideration of 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 [2]. The yield strength for ties, #3 bars, and concrete reinforcing for stirrups was 275.8 MPa (40,000 psi). The yield strength for all other deformed bars and all welded wire fabric was 413.7 MPa (60,000 psi). The 28-day concrete compressive strengths included 20.7 MPa (3,000 psi) foundation and equipment bases and 27.6 MPa (4,000 psi) 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.

Plans showed that the design live loads followed requirements of the Oklahoma Building Code. Wind loads were included but no blast or earthquake loads were required.

Concrete and reinforcing bar samples were obtained from materials moved to the Oklahoma County Sheriff's Firing Range. During this visit, the BPAT reviewed photographs taken soon after the explosion by several law enforcement organizations. Also, several pieces of building debris were inspected.

Analysis of Materials and Determination of Blast Size—Five of the cores taken from the concrete from the Murrah Building were selected for compression testing. Compression test results indicated that the concrete strength was well in excess of the 27.6 MPa (4,000 psi) called for in the design specifications. Petrographic evaluation indicated that the concrete contained normal-weight aggregate and was of the quality required in the design specifications.

Several pieces of reinforcing bars were recovered from the debris of the building. A few lengths of straight bar were tested in tension. 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.

Using forensic engineering techniques, it was possible to determine the blast size. These findings provided a means for assessing response of the building and potential mitigation techniques. Soil test boring data obtained along N.W. Fifth Street was found in the architectural plans. The borings were taken approximately 5.9 m (19.5 ft) south of the street centerline. Crater survey measurements located the center of the crater to be approximately 8.5 m (28 ft) south of the street centerline.

During the crater survey, pavement thickness was measured to include approximately 275 mm (11 in) of asphalt over 175 mm (7 in) of concrete at the north tip of the crater. Borings provided the soil properties below the concrete layer.

Based on observations made of the crater and other damage, the blast that damaged the building had a yield equivalent to approximately 1814 kg (4,000 lb) of TNT [2]. This extremely large explosion was centered approximately 4.8 m (15.6 ft) from Column G20 (see fig. 2 and fig. 3). The blast caused a crater approximately 8.5 m (28 ft) in diameter.

While the north face of the building sustained the brunt of the effects of the blast, structural damage to the remaining exposures was more limited. Most of the damage was caused by progressive collapse following loss of three columns nearest the blast.

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.

3.0 PROBABLE FAILURE MECHANISM

A forensic investigation of damage to the Murrah Building disclosed that failure occurred in three columns supporting a transfer girder on the north side of the building. One of these columns was destroyed by brisance or shattering while the adjacent two failed in shear. After losing support from the three columns, approximately 50 percent of the floor area of the building collapsed, producing a large number of casualties.

A photo of the Murrah Building prior to the terrorist attack is shown in figure 1. The transfer girder supported on four intermediate columns can be seen just below the glass curtain wall.

Figure 3 shows a drawing of the building north elevation with the explosive-laden truck parked in the street beside it. Also shown in the figure is the resulting crater. Figure 4 shows a plan view of the crater and location of the column nearest the truck. As reported in reference 1, a bomb having an explosive power equal to approximately 1814 kg (4,000 lb) of trinitrotoluene (TNT) was detonated in the truck.

Analysis of damage in the field and calculation of effects of the explosion determined that the nearest column, G20 as identified in figure 2 and figure 3, would have been destroyed by brisance or shattering. Approximately 12.2 m (40 ft) each side of the column that was destroyed, adjacent columns were found to have exceed their shear strength, but been approximately at their flexural capacity. As a result, the adjacent columns failed in shear, thereby leaving the spandrel beam without support for distance of approximately 48.8 m (160 ft).

Transfer girder reinforcement is shown in figure 5. Three of the transfer girder top bars were continuous but none of the bottom bars were. A similar pattern was present in spandrel girders above the third floor as shown in figure 5. Consequently, there was no effective integrity reinforcement in the spandrels of this building. It must be noted that at the time the building was built, no integrity reinforcement was required. Analyses show that this building met or exceeded code requirements in every way that was checked.

Reference 1 indicated that shear failures of two of the three columns could have been prevented if only a small amount of hoop steel had been provided. It is also possible that the column nearest the bomb could have been saved if full confinement reinforcement had been provided. However, for purposes of this discussion, it will be assumed that the column nearest the truck would be destroyed by brisance.

In reference 1, an analysis is made with one column removed. The three mechanisms that could develop after removal of one column are shown in figure 6. Calculations indicate that Mechanism 2 results in a capacity of approximately 2.9 kPa (60 psf) to 3.4 kPa (70 psf) floor above the spandrel beam. This is significantly less that the approximately 5.3 kPa (110 psf) of self-weight plus live load that existed in the building. Consequently, removal of any one column supporting the as-built transfer girder would cause failure of all of the floors above over a length of about 24.4 m (80 ft).

4.0 APPLYING EARTHQUAKE DETAILING TO REDUCE LOSSES

Ordinary Moment Frames have limited capacity 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 2002 edition of *NEHRP (National Earthquake Hazards Reduction Program) Recommended Provisions for Seismic Regulations for New Buildings*, provide structural systems with much higher ability to dissipate energy. Special Moment Frames are detailed with continuity of top and bottom reinforcement, shear strength to resist maximum probable moments, and confinement at potential hinging locations. It is noted that the NEHRP recommendations for design of Special Moment Frames and Dual Systems were not widely available until 1985, approximately 10 years after the Murrah Building was constructed.

If detailing currently used 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. With these columns in place, a mechanism consisting of a combination of vierendeel, catenary, and transfer girder flexure can develop to minimize the loss of structure.

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 were 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 were removed by the blast, 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 destroyed 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 were removed by the blast and failure occurred in 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.

4.1 Integrity Reinforcement

Since 1989, requirements for structural integrity reinforcement have been listed in Section 7.13 of the ACI Building Code [3]. In general, Section 7.13 requires one-sixth to one-quarter of the reinforcement in perimeter beams to be continuous about the building. In addition, Section 13.4.8.5 requires two bottom bars in each direction be carried through columns continuously or anchored within the columns.

The purpose of integrity reinforcement for both spandrel beams and slab column intersections, is to provide a small capacity, even after failure of a column or slab at any one location.

In Chapter 21, toughness is required throughout the structure for seismic resistance. to obtain required toughness, Section R21.3.2 states the following:

> "Lap splices…..are prohibited at regions where flexural yielding is anticipated because…..splices are not reliable under…..cyclic loading into the inelastic range."

Based on this analysis, it is concluded that use of Special Moment Frame detailing has the potential for significantly improving blast resistance.

4.2 Full Capacity Mechanical Butt Splices

In the 1999 ACI Code [4], full capacity mechanical butt splices were recognized for the first time. Classified as Type 2 mechanical splices in Section 21.2.6.1(b), they are required to "develop the specified tensile strength of the spliced bar." ACI 318-99 permits use of Type 2 splices at any location, including hinging regions. Consequently, Type 2 full capacity mechanical butt splices can be used to connect integrity reinforcement and improve the blast and/or seismic resistance of concrete structures.

If integrity reinforcement in the spandrel beams of the Murrah Building was maximized by making all of the bars continuous with full capacity mechanical butt splices, significant increase in the capacity of Mechanism 2 of figure 6 would be realized [5]. Using Mechanism 2 with 100 percent of the bars spliced with full capacity mechanical butt splices, calculated capacity of the building would be at least 6.7 kPa (140 psf). Unit weight of the building, including some live load, is approximately 5.3 kPa (110 psf). Consequently, even if one column had been removed the building would not have collapsed.

Although collapse of the building frame would have a high probability of being prevented by making all of the reinforcement in spandrels continuous, several of the floors would be destroyed as a result of brisance. Consequently, the reduction in catastrophic damage to the building would not be 100 percent, but is estimated to be approximately 80 percent.

As indicated in reference 1, almost all of the casualties were the result of building collapse, not air blast. Consequently, if building collapse is reduced by 80 percent, casualties would be reduced by a similar amount.

4.3 Cost Analysis

To determine what the additional cost might be if full capacity mechanical butt splices had been used in the Murrah Building, a review of cost studies done by Cagley and Associates and reported in reference 5 was made.

Cost analyses done by Cagley and Associates [4] show that the increased cost of mechanical butt splices over lap splices in a 12-story parking deck was less than ¼ of 1 percent of the cost of the building. In an office structure, such as the Murrah Federal Building, the cost of mechanical splices would be even a smaller percentage of the total cost. Considering that use of full capacity butt splices could be limited to only that portion of the building exposed to the street where the bomb was detonated, estimated total cost differences are approximately 1/8 of the total cost of the building. This cost difference is insignificant.

5.0 SUMMARY

This paper discusses the potential for using earthquake resistant detailing and mechanical full capacity butt splices to increase blast and earthquake resistance of buildings. With seismic detailing, it is shown that damage due to blast can be significantly reduced. The need for integrity reinforcement to reduce damage in a building when unanticipated intense loads destroy a single element is also discussed.

Terrorist bombing of the Murrah Federal Building in Oklahoma City is used as a case study. It is shown that without fully continuous integrity reinforcement, strengthening of columns with seismic detailing could reduce damage by about 50 percent. However, if full capacity mechanical butt splices had been used to make all of the spandrel beam reinforcement continuous, collapse of the building would have been reduced by an estimated 80 percent with a similar reduction in casualties. The estimated additional cost to provide this continuous reinforcement is approximately 1/8 of 1 percent, much less than the variation in estimating the cost to construct the building.

6.0 REFERENCES

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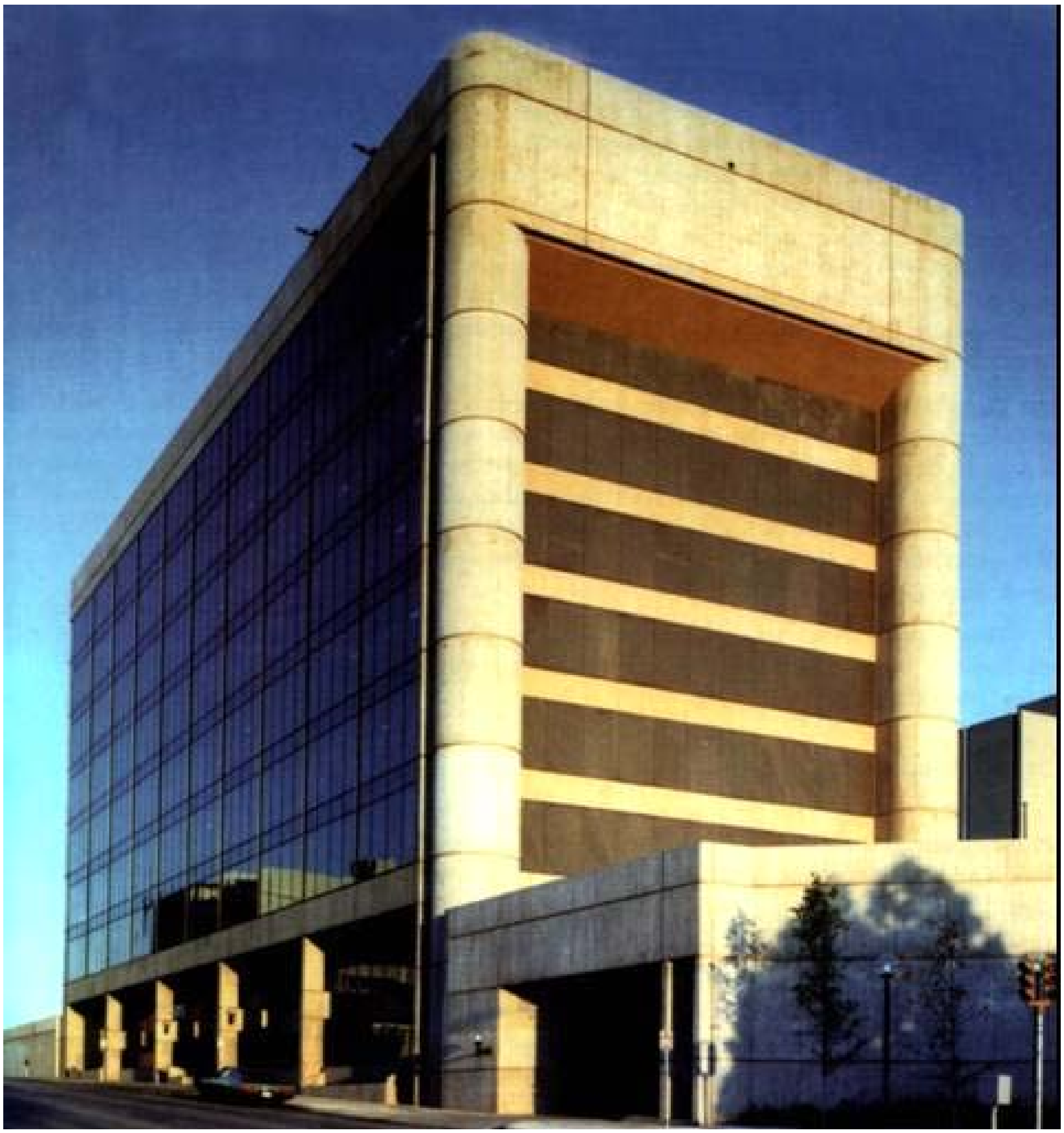


Figure 1. Murrah Building Prior to Blast.

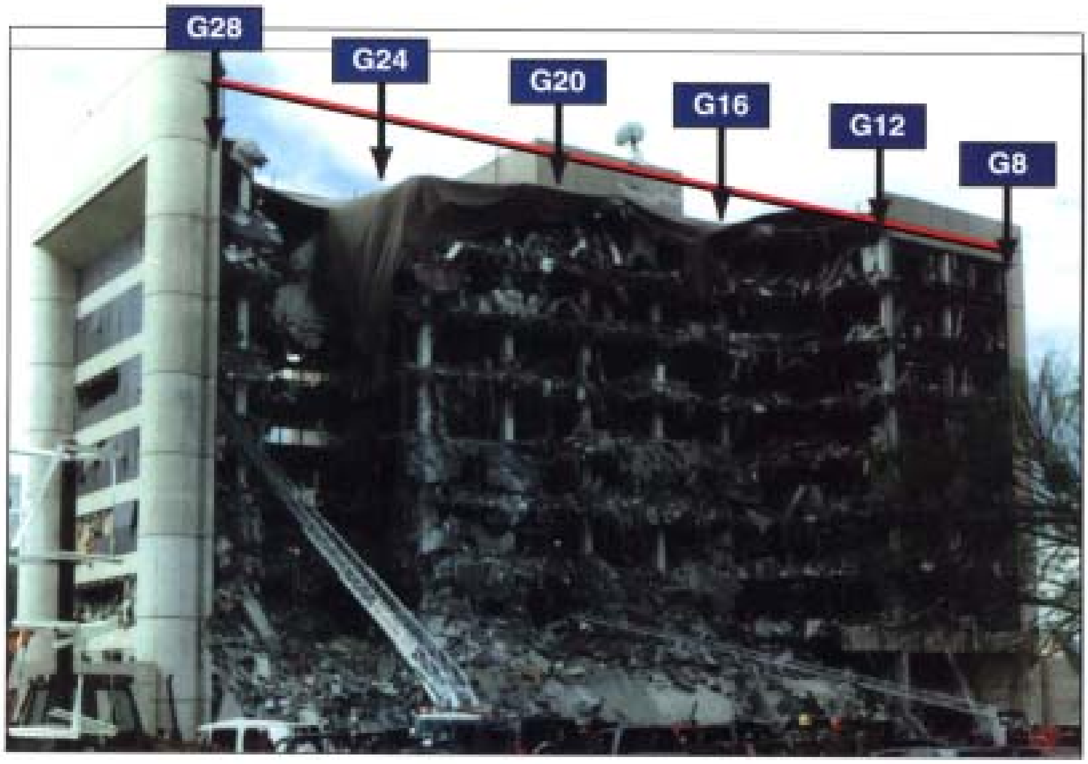


Figure 2. Damage to North and East Sides of Murrah Building

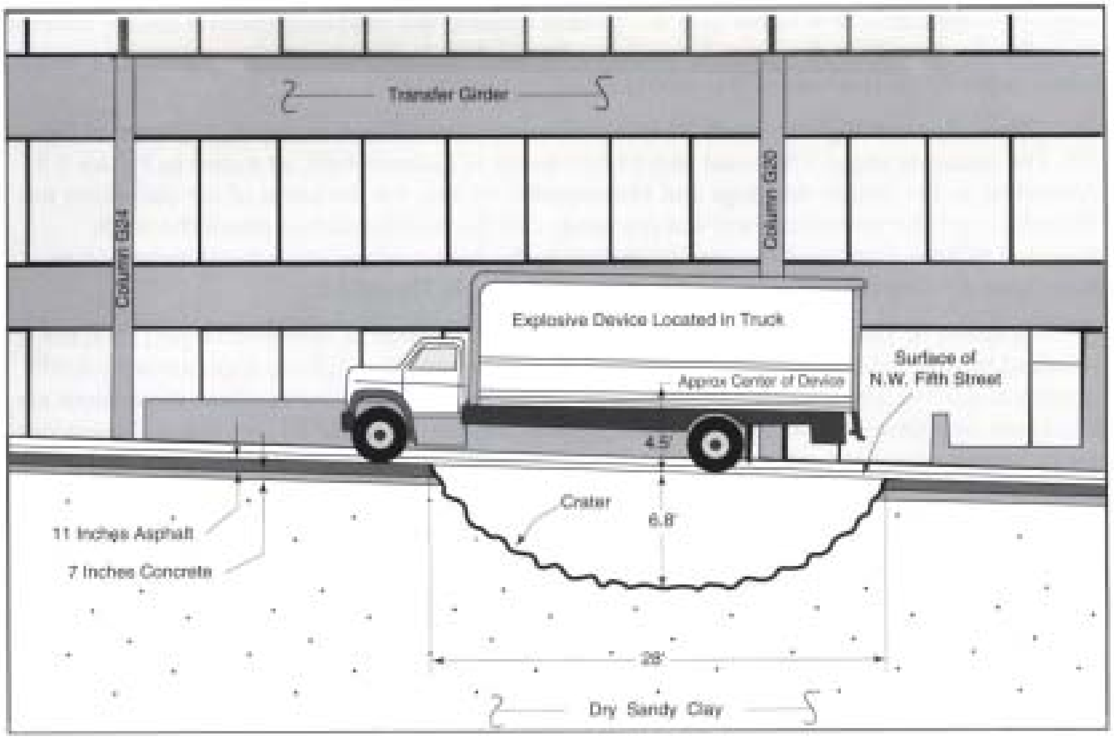


Fig. 3 Approximate Dimensions of Crater at North Face of Murrah Building

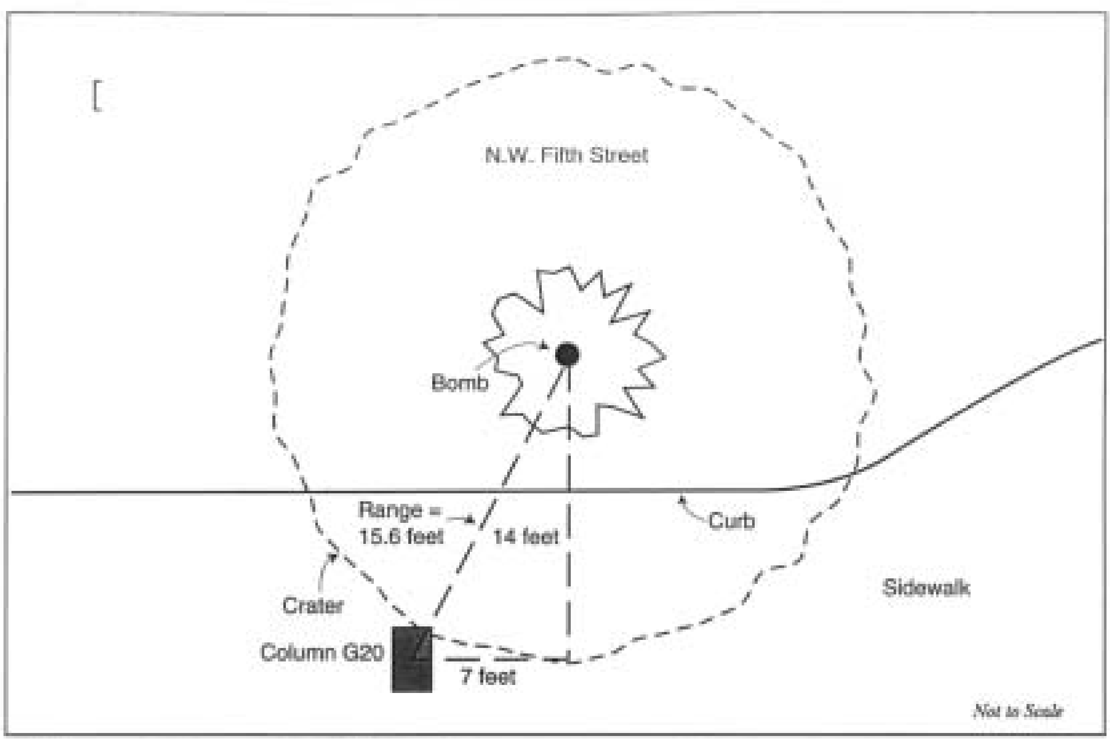


Figure 4. Proximity of Column G20 to Location of Bomb (Plan View).

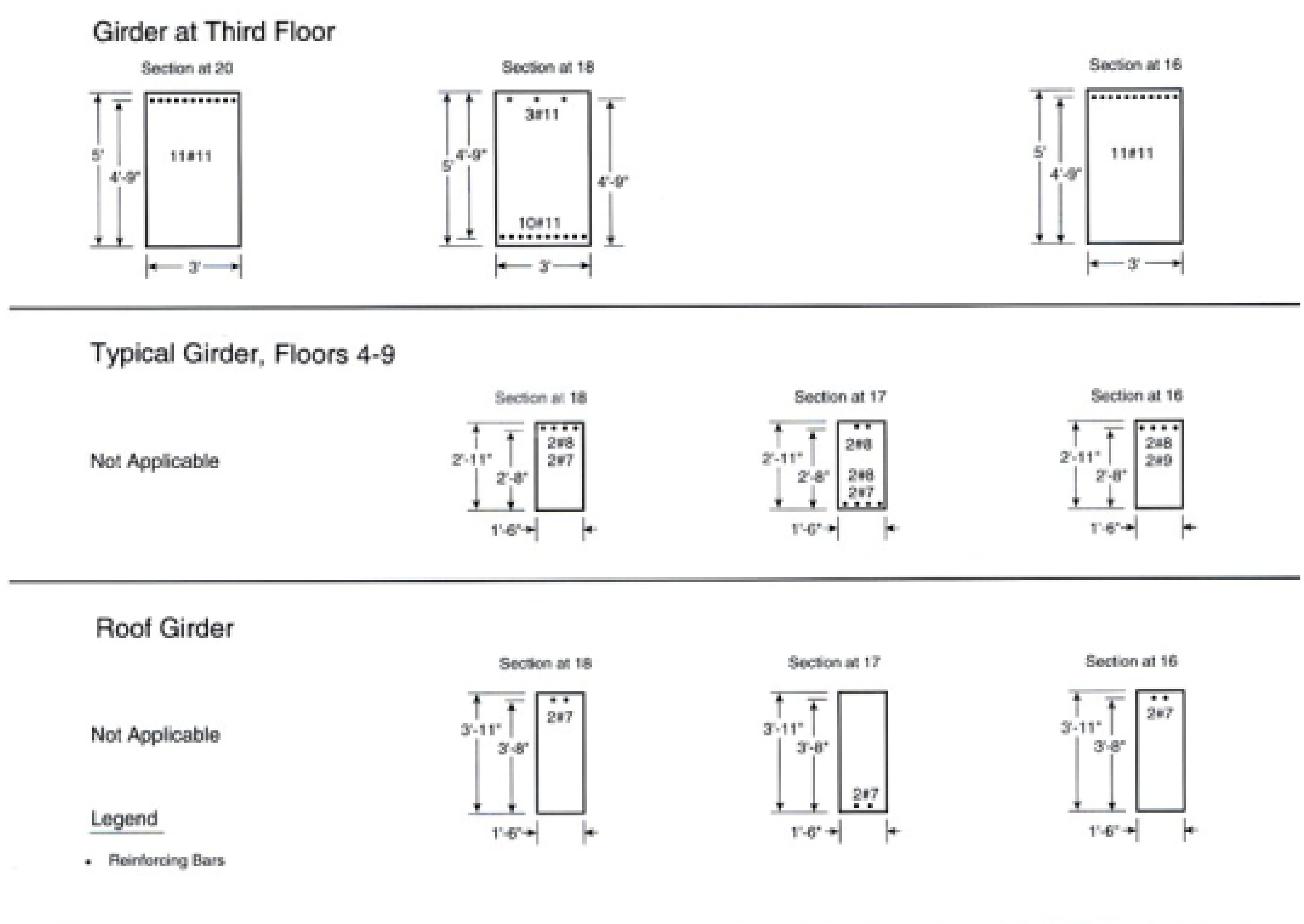
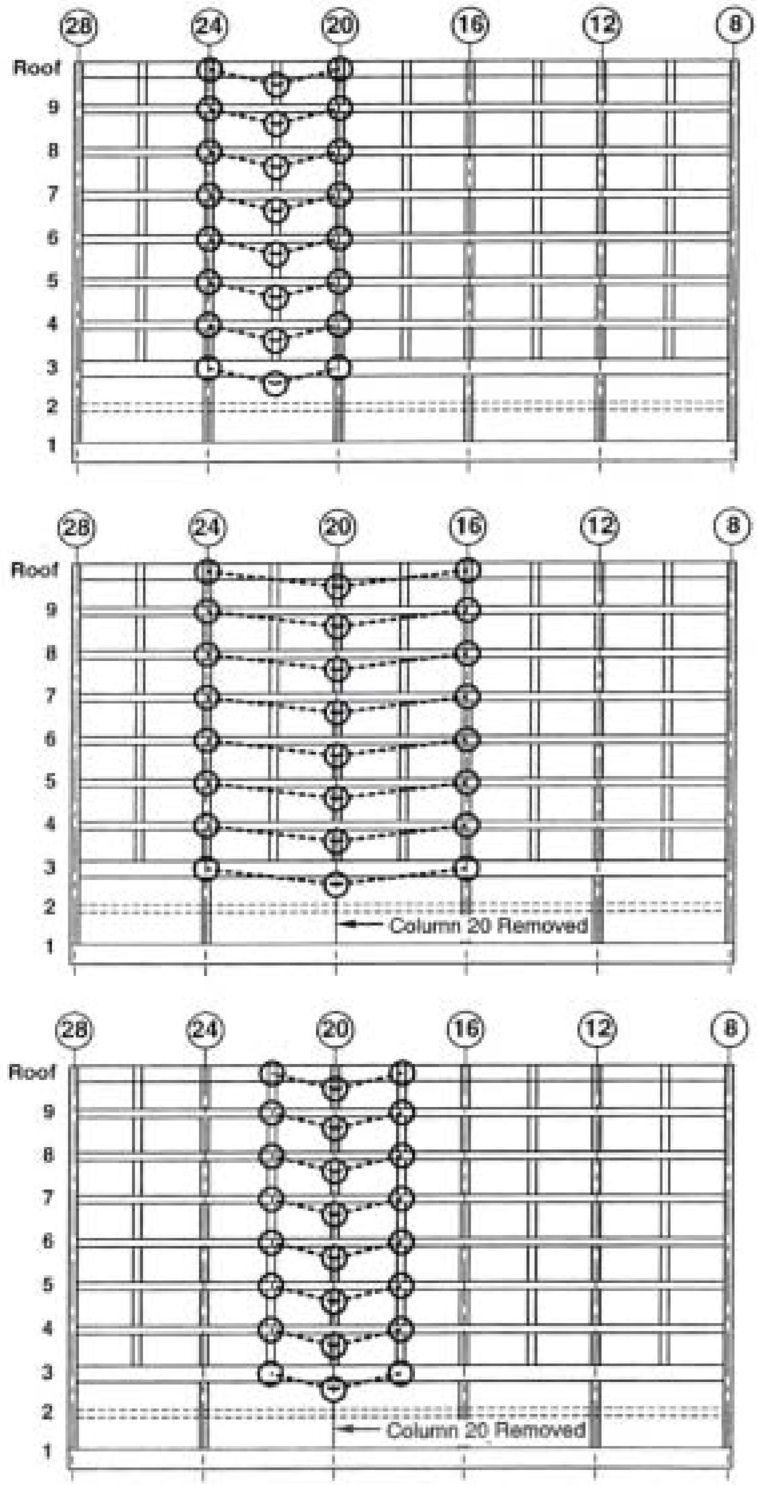


Fig. 5 Cross-Sectional Distribution of Girder Reinforcing Bars.



Mechanism 1

Mechanism 2

Mechanism 3

Figure 6. Possible Mechanisms with No Columns Removed and One Column Removed.