Blast Loading and Response of Murrah Building

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Abstract

The truck bombing of the Murrah Building caused significant damage to this structure. From the characteristics of the bomb crater, the explosion yielded energy comparable to that from the detonation of 4,000 lb of trinitrotoluene (TNT). This directly removed a principal exterior column and the associated airblast failed two others. The airblast also destroyed some of the floor slabs in the immediate vicinity.

Introduction

The authors represented ASCE on the Building Performance Assessment Team (BPAT) which investigated the collapse of the Murrah Building. In the report of this work (BPAT, 1996), the design, construction, and condition of the Murrah Building prior to the tragic bombing has been described in detail. The structure was a nine story reinforced concrete frame with three rows of columns spaced at 20 ft. A large transfer girder at the third-floor permitted the elimination of alternate exterior columns below. The building was designed and constructed in accordance with the applicable codes, but did not provide any deliberate resistance against a vehicular bomb attack.

This paper describes the estimation of the blast loading and its direct effect on the structure of the building. First, the blast loading is inferred from the properties of the crater formed by the explosion. Then the response of critical

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structural elements to this loading is computed using approximate methods appropriate to the purpose of the assessment. These include the principal exterior columns supporting the transfer girder and the floor slabs of the building.

In the BPAT report (1996), the further implications of these direct responses are examined. Of particular interest is the integrity of the structural frame with these damaged columns. Measures to mitigate this situation in other buildings are also discussed.

Blast Loading

The calculation of the blast loading begins with the estimation of the yield or quantity of explosives detonated. For bursts near the ground surface, this is usually inferred from the dimensions of the crater formed. The engineering survey of the crater formed the basis of this deduction for the assessment team.

The crater, Figure 1, was approximately 28 ft in diameter and 6.8 ft in depth. According to the design drawings and observations on site, the thickness of the pavement was 18 in. 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 ft above the ground surface.

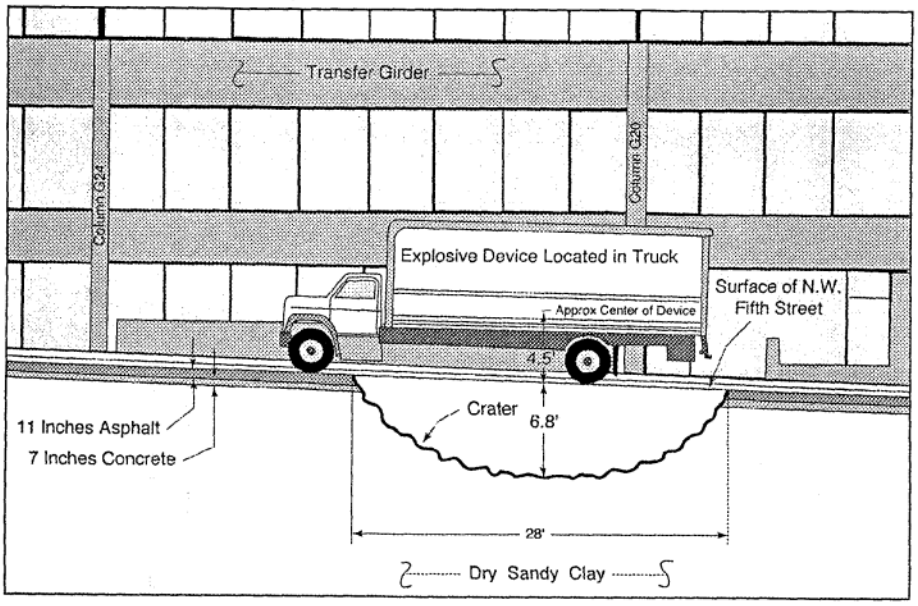

Figure 1. Bomb Crater at North Face of Murrah Building

As summarized in Table 1, the detonation of TNT having a mass of 4,000 lb results in a crater whose dimensions are consistent with those measured at the Murrah Building. The equivalent mass of TNT is used here, as is the practice in explosive effects, as a measure of the energy yielded by the bomb. In this analysis, the crater dimensions for pavement on soil are an average of those for massive concrete and for those on dry sandy clay alone. The individual dimensions were computed from empirical equations fit to extensive test data (Department of the Army, 1986). These were weighted in proportion to the depths of the two materials in the crater. This is substantiated by the results of ongoing research concerning craters in pavements with an underlying soil base.

Table 1. Estimate of Bomb Yield from Crater Dimensions

Condition	Depth ft	Diameter ft
4000 lb TNT Massive Concrete	2.6	13
4000 lb TNT Sandy Clay	8.5	31
4000 lb TNT Pavement on Soil	7.2	27
Engineering Survey of Murrah Building	6.8	28

Given this charge size, empirical relations have been developed for the airblast waveform as a function of range and orientation based on experimental and analytic evidence (Department of the Army, 1986). These indicate that the detonation imposed a severe airblast loading on the north face of the Murrah Building, which was only 14 ft away. The peak overpressure ranged from over 10 ksi at the closest point to 9 psi at the upper west corner, with an equivalent uniform value of 140 psi. The duration of this loading was limited and had an equivalent uniform value for a triangular pulse of 5 msec.

Response of Adjacent Column

Column G20 was the exterior column supporting the noteworthy transfer girder that was closest to the explosion. In fact, it was at the edge of the crater only 14 ft south and 7 ft west of the center. This corresponds to a scaled range

$$H/W^{1/3} = (14^2 + 7^2)^{1/2} / 4000^{1/3} = 1.0 \text{ ft/lb}^{1/3}$$

in which H is the horizontal range in feet and W is the explosive energy expressed as an equivalent mass of TNT in lb.

Experience indicates that adjacent explosions destroy reinforced concrete columns by the direct or shearing effects of blowing out, severing, and undermining. Bomb damage assessments following World War II reported that this occurred within scaled ranges of 3.0 ft/lb$^{1/3}$ for cased bombs (National Defense Research Committee, 1946). Based on contemporary research on the breaching of concrete walls (McVay, 1988), the corresponding limit for brisant failure of columns by bare charges is estimated to be 1.5 ft/lb$^{1/3}$.

Thus in all likelihood, Column G20 was abruptly removed by brisance following the explosion. At a scaled range of 1.0 ft/lb$^{1/3}$, it was well within the experimentally based range for this phenomenon. Further, no one found any evidence of this column in the debris or in the crater following the bombing.

Response of Nearby Columns

Column G24 was located outside the range of brisance, but was highly loaded by the detonation. As indicated in Figure 2, its dynamic response to this load is approximated as a single-degree-of-freedom, simply supported beam between the first-floor and third-floor elevations (Biggs, 1964). The 36 in. by 20 in. column resisted this loading about its weak axis. The strength was limited by the shear resistance at the ends of the span. From the material properties measured in this study and the axial prestress estimated for dead and actual live loadings, this limiting static capacity, V_u, corresponds to 52 psi uniformly distributed on the 36 in. face.

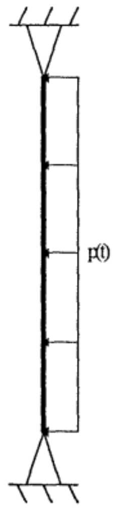

Figure 2. Analytical Model of Nearby Columns

Figure 3 shows the blast loading on Column G24. On the front face, the peak load p_o rises abruptly to the reflected pressure, 1,400 psi. 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 t_d of the net loading is only 1.3 msec.

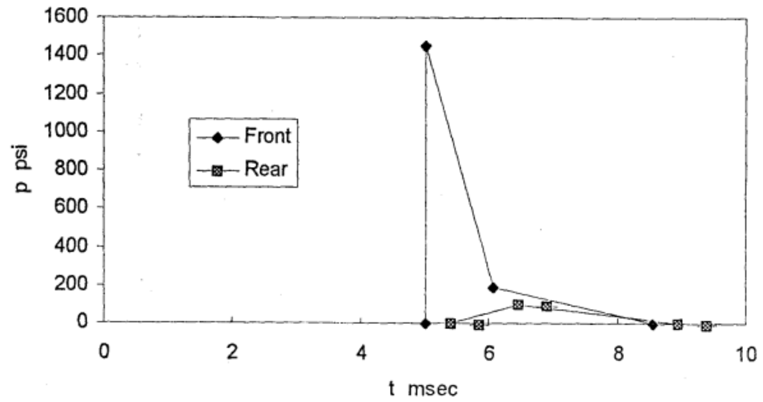

Figure 3. Blast Loading of Column G24

Most of the response of Column G24 occurred after the net load diminished to zero so that it was an impulsive structural event. In this response the maximum shear V_m induced at the supports is 1.8 times V_u and a brittle failure of the element ensues. 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, this column was missing from above the first-floor, in agreement with the results of this analysis.

As presented in Table 2, the slant range R to the midheight is greater for Column G16 than for G24. At this distance of 50 ft, the peak pressure is still 641 psi. According to an analysis similar to that performed for Column G24, the response just reaches the shear capacity. This implies an incipient brittle failure which is consistent with the condition of this element after the explosion.

Table 2. Blast Response of Intermediate Columns Supporting North Transfer Girder

Column	R ft	p_o psi	t_d msec	V_m/V_u
G24	37	1400	1.3	1.8
G16	50	641	1.7	1.0
G12	89	115	1.4	0.1

Column G12 did endure the direct blast effects of the bombing. It was located at a slant range of 89 ft as indicated in Table 2. Here, the loading was 115 psi. 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.

Response 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 ft by 10 ft glass panels restrained by aluminum channels which offered insignificant resistance to the propagating blast wave. The filling pressures below the slab were greater than those above and caused an upward loading on each slab.

This net upward loading is considered as a spatially uniform pressure for the purposes of this assessment. The 6 in. slab is dynamically modeled as a single-degree-of-freedom, simply supported element spanning from east to west between beams at the floors and roofs (Biggs, 1964). This element was competently detailed for the downward dead and live loads with reinforcement in the bottom of the slab. However, this arrangement provided only an incidental resistance of 0.24 psi against the upward action of the blast.

Figure 4 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 the slab. These loadings are further represented by triangular pulses as shown. In this particular case, the load from below has a peak of 154 psi while that from above is only 87 psi. In both cases, these loads act for relatively short durations.

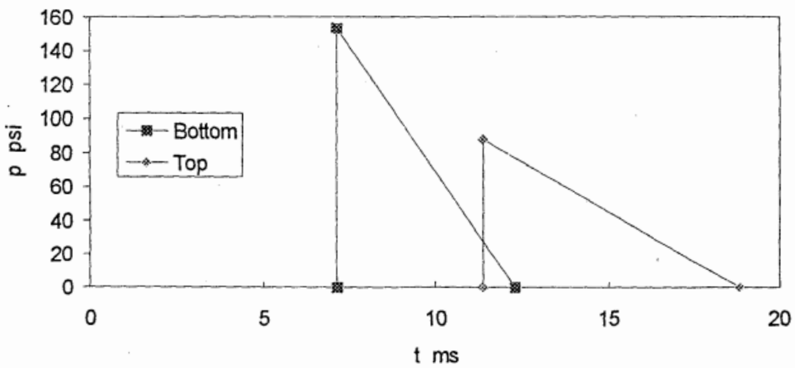

Figure 4. Blast Loading of Illustrative Slab

The calculated response of this slab must consider the static dead and actual live loads in addition to the blast in Figure 4. The dynamic response has a long period and the blast event represents an impulsive loading condition. In the particular case illustrated, the upward deflection exceeds the ultimate capacity of the floor slab and the slab collapses.

Similar analyses were performed for the other floor and roof slabs in the building and are summarized in Figure 5. 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. The inward extent of this directly induced failure was estimated to be 40 ft at the second-floor and to diminish to zero at the sixth-floor. However, the other slabs responded elastically to the differential blast loading and in some cases were not loaded above the static downward loads.

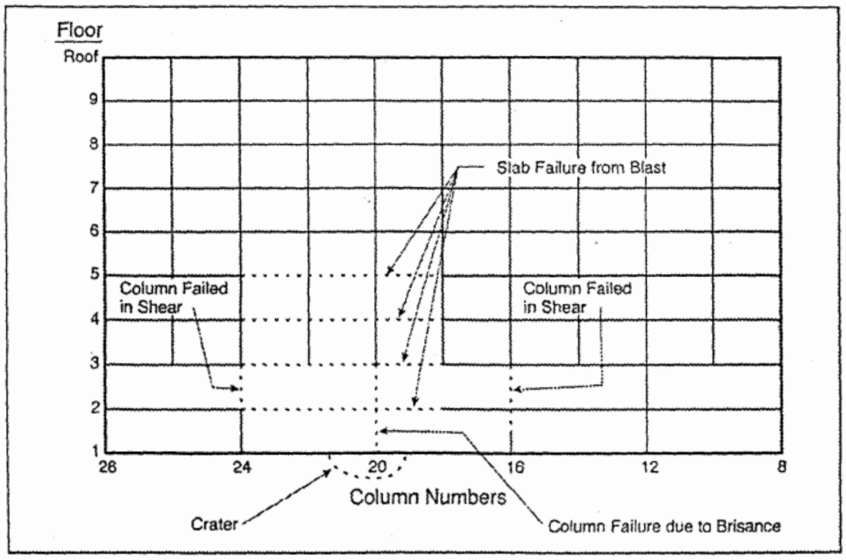

Figure 5. Blast Response of Slabs

Conclusions

The Murrah Building suffered important structural damage as a direct result of the tragic bombing. This included the failure of three intermediate principal columns supporting a third-floor transfer girder. Some floor slabs in the proximity of the bomb were also directly demolished.

Acknowledgments

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