Ground Motions and Air-Blast Effects of Explosive Demolition of Structures

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Abstract: Ground motions and air-blast overpressures produced by the explosive demolition of about 27 structures were compared to determine the importance of these effects. To provide background for this demolition method, general design principles were presented along with details of the demolition of a 12-story reinforced-concrete building and a steel girder bridge. Trends in the observed ground- and air-borne disturbances were compared to those expected from rock-blasting and drop-weight impact. Air-blast overpressures can be atypically high, and building impact typically produced greater motions than the explosions.

INTRODUCTION

This paper lays the foundation for an evaluation of ground- and air-borne disturbances produced by the explosive demolition of structures. This evaluation is based on the comparison of about 27 case histories worldwide, which present sufficient blast design information and measured environmental response. The paper begins with sections on general demolition blast design, which includes detailed cases for a 12-story office building and a steel girder bridge. Case histories are then synthesized and compared with levels of disturbance expected from rock blasting. Trends in the data are then compared to those predicted with two degree-of-freedom models of falling-weight impact.

Explosive demolition is not new, given its long-standing importance in military engineering. What is new is its increasing popularity for razing multistory office buildings in urban areas. This popularity is the result of pioneering work of several contractors around the world, greater control of detonation initiation sequences, and development of shaped charges.

More than 130 articles that reported on the explosive demolition of structures were found; only five articles reported sufficient information for comparison. These five articles contained 27 cases, but only one was from the United States. Despite numerous attempts to contact municipal officials associated with reported explosive demolition events, only this one case was developed. It appears that few measurements are made in the United States and details of blast design and structural crippling are not widely reported for fear of loss of competitive advantage.

Though this paper would be appropriate for publication in either a structurally or geotechnically oriented medium, a construction focus was chosen, because demolition is primarily a construction rather than design or analysis activity. The writer hopes this paper will allow regulatory officials to assess the need for basic data to understand possible consequences, and lay the groundwork for the development of guides to practice this useful construction technique.

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Note. Discussion open until May 1, 1995. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this paper was submitted for review and possible publication on April 14, 1993. This paper is part of the ***Journal of Construction Engineering and Management***, Vol. 120, No. 4, December, 1994. ©ASCE, ISSN 0733-9364/94/0004-0838/$2.00 + $.25 per page. Paper No. 5985.

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BLAST DESIGN

Design for explosive demolition must include equal consideration of structural stability and explosion initiation and effects. Although this paper focuses on environmental effects of explosions, the importance of structural engineering in explosive demolition cannot be overemphasized. Failure to destabilize the structure may be far more dangerous than exceeding conservative environmental controls. Such failure can occur because of a lack of understanding of structural behavior. In other words deciding: (1) Which members to precut to reduce the unloaded factor of safety to 1.000001; and (2) the order to begin explosive cutting is as important as knowing how to explosively sever the chosen members. However, such structural decisions are unique to each structure and are beyond the scope of this paper.

The discussion of blast design is presented to assess the general size of the charges and the initiation sequence to be employed on typical projects. Empirical rules for general design will be presented along with two case histories to document specific charge weights. These general guides and the two cases represent only a fraction of possible approaches and should not be interpreted as definitive, but rather as illustrative.

Four basic building elements need to be severed in order to demolish a structure—mass concrete, reinforced-concrete walls, reinforced-concrete beams and columns, and steel girders and columns. Examples are presented to illustrate the size of explosions to determine their vibration and air blast significance. Further details of the explosive design can be found in the original sources and Dowding (in press, 1995).

Disintegration of mass concrete follows rules designed for rock excavation in which powder factor and hole spacing are sufficient for design, as shown in Table 1 (Gustafsson 1981). The powder factor is the explosive energy per volume of fragmented material and is typically defined as weight of explosive per volume of concrete. Walls and columns (with and without reinforcing) can be disintegrated, provided the shot parameters, defined in Table 2 (Gustafsson 1981), are maintained.

Disintegration of a typical 0.5 by 0.5 m (1.6 by 1.6 ft) square, 4-m-high, reinforced-concrete column would require two rows of holes with 0.085 kg charge per hole, with a hole spacing of 0.35 m. If all holes were initiated at the same instant, ($2 \times 4/0.35 = 23$ holes) $\times$ (0.085 kg per hole) = 1.9 kg (4.3 lb) of explosive would be detonated. This weight could be consid-

TABLE 1. Specific Charge (per unit vol.) for Disintegration of Massive Concrete

Type (1)	Specific charge[a] (kg/m^3) (2)	Hole spacing[b] E_1 (m) (3)
Nonreinforced concrete of poor quality	0.25–0.30	0.80–0.90
Nonreinforced concrete of good quality and material strength	0.30–0.40	0.75–0.90
Reinforced concrete with heavy reinforcement	0.80–1.00	0.55–0.60
Extra powerful reinforced concrete of military type	1.50–2.00	0.4–0.50 0.50–0.55

[a]Distributed equally along holes with prepackaged, thin tube units with 0.32 kg explosive per meter of hole

[b]Square pattern

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TABLE 2. Hole Geometry and Loading for Disintegration of Concrete Columns

Width (largest) (m) (1)	Depth of hole (m) (2)	Hole spacing E_1 (m) (3)	Number of hole rows (4)	Charge (per hole) (kg) (5)
0.30	0.20	0.30	1	0.05
0.40	0.30	0.30	1	0.10
0.50	0.40	0.35	2	0.085
0.60	0.45	0.35	2	0.125
0.70	0.55	0.35	2	0.17

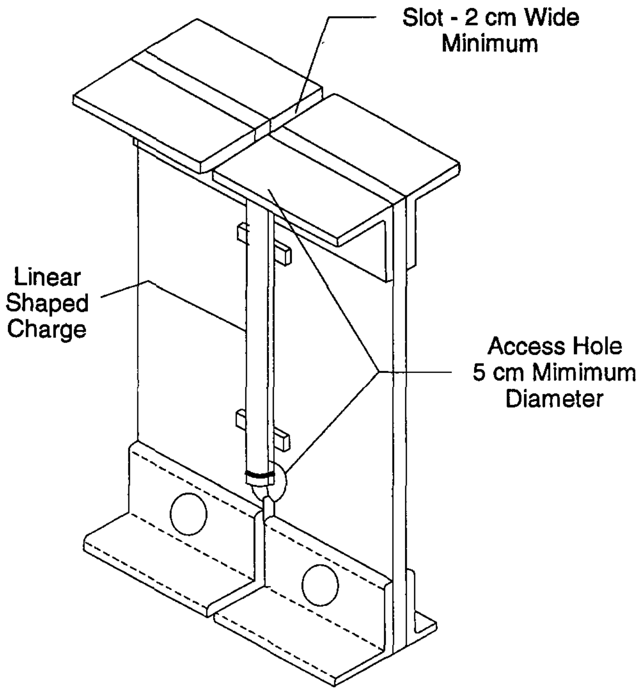

FIG. 1. Preparation Details for Explosive Cutting of Steel Girder

erably reduced by severing only 1 or 2 m of the column, or alternatively severing the reinforcing after chipping away the concrete and blocking the column. However, such chipping and blocking does not seem to be practiced as it may not be practical, because of the loss of moment resistance.

Exposed steel girders are cut with small linear shape charges, shown in Fig. 1 (from Flagg 1976) (precut flanges, web access holes, and placement of shaped charge across web, between flange cuts are shown). It has been shown (Barbour 1981) that a 12 mm (0.5 in.) mild steel plate can be severed by shape charges, with explosive weights as small as 300 grains per ft (1 grain is equivalent to 0.065 g) per foot (64 g/m). Thus the web of a 12 WF 75 girder could be severed by only 0.05 lb of explosive (300 grains = 19.5 g = 0.043 lb).

Case studies are presented to illustrate the importance of blast initiation

FIG. 2. Directional Collapse Patterns for Beam and Column Building and Steel Girder Bridge: (*a*) Directional Building Collapse in Mexico City; (*b*) Uniform Bridge Drop over Mississippi River (Barbour 1981)

timing and the preexplosion removal of key load-bearing members. More importantly, they reveal the total explosive weights likely to be employed as well as the maximum detonated at any instant (per delay).

Fig. 2 compares the collapse geometry of a reinforced-concrete office building in Mexico City and a steel truss bridge in Mississippi. The building is shown to collapse from right to left, as a result of initiating column detonations beginning on the right and proceeding to the left. Such sequential initiation directs the building's fall to the right, and further disintegrates the components by subjecting them to increased shearing. Alternatively, the entire bridge span is dropped evenly, which indicates the charges were all initiated at the same instant, rather than sequentially. Although the span could have been dropped by cutting only at the abutments, it was coincidentally cut into sections small enough to be recovered by a barge, without additional cutting in the water.

DEMOLITION DETAILS FOR REINFORCED-CONCRETE COLUMN STRUCTURE

Specific blast design information was reported in an unusually detailed fashion for the 12-story reinforced-concrete structure shown in Fig. 3 at No.

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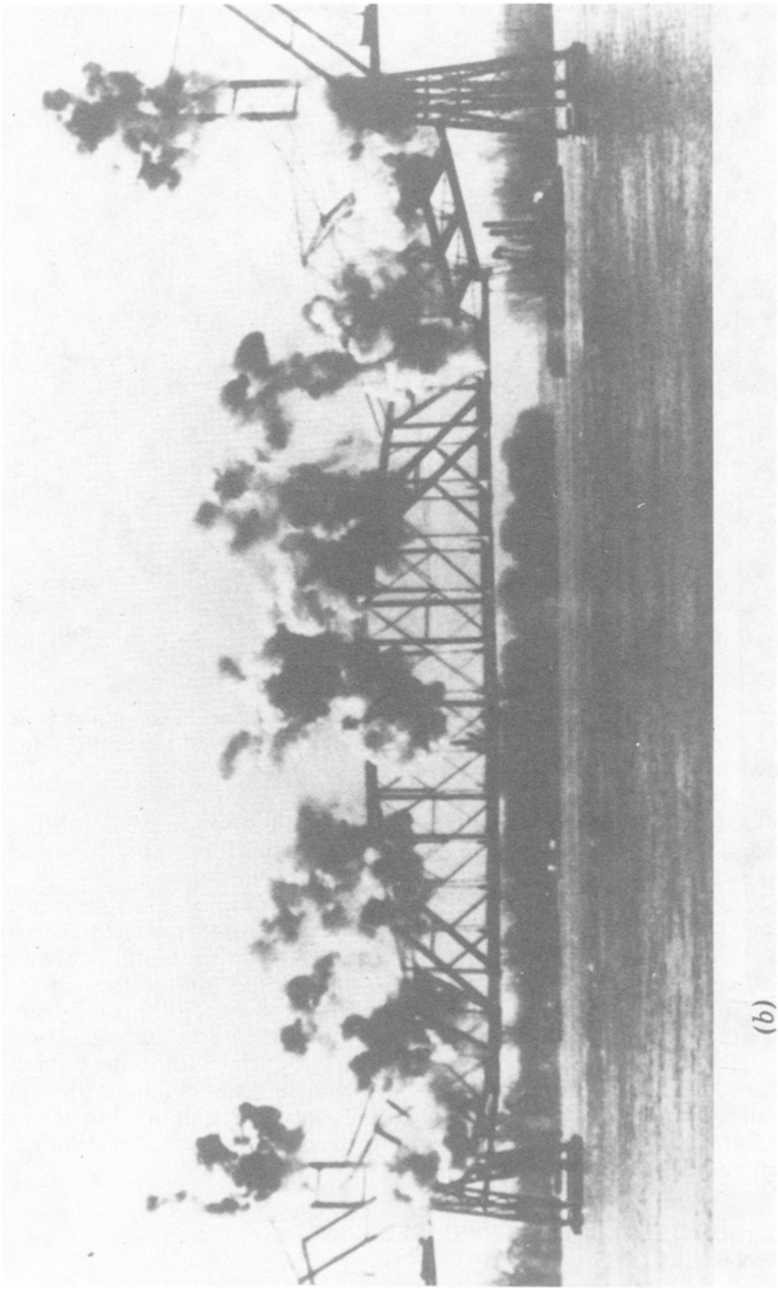

(b)

FIG. 2. (Continued)

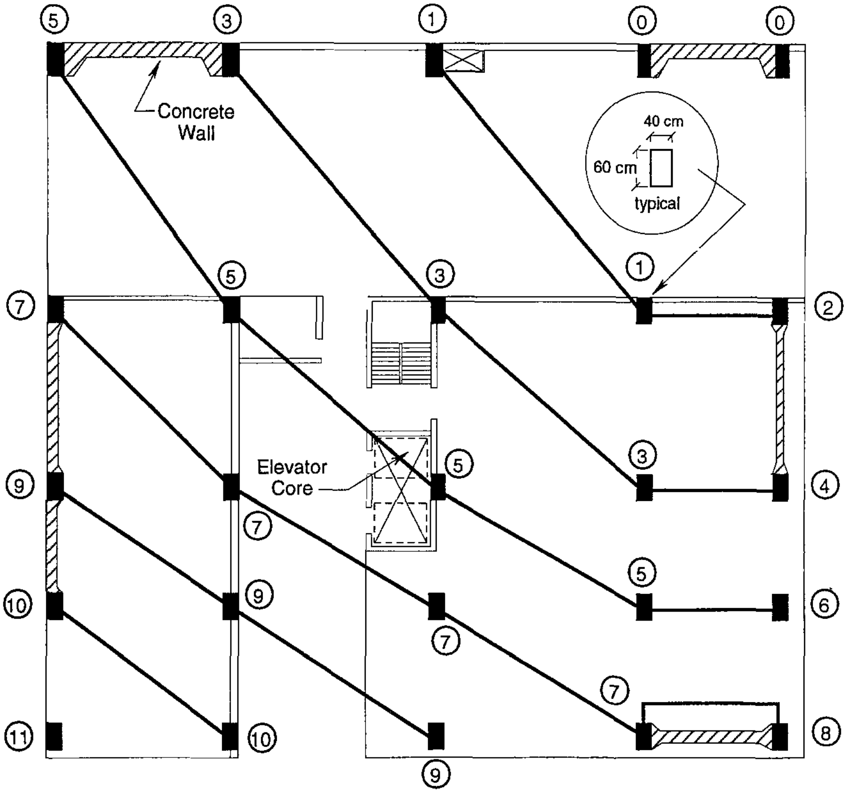

FIG. 3. Column Location and Explosive Initiation Sequence for Monterrey Building

158 Monterrey, Mexico City (Olavarrieta, personal communication, 1987), Mexico. In September 1985 the Mexican earthquake severely damaged, but not to the point of collapse, scores of medium-height buildings in downtown Mexico City. Consequently the Monterrey building and more than 30 other severely damaged structures were brought down explosively, and data obtained during that exercise provide the majority of detailed information available regarding explosive demolition (Contreras and Leon, unpublished report, 1988; Olavarrieta 1986).

Explosive demolition was preceded by the removal of all interior and shear walls by hand. Other structurally stabilizing elements such as stairway ramps and elevator shafts were also removed before the columns. The remainder of this paper will concentrate on column removal, which controls the blast-initiation sequence.

Proximity of the adjacent structures forced the collapse to be designed to topple toward the northeast (upper right) corner, into the street. Such directionality was ensured by initiating the explosive sequence at the corner toward which the building is to topple. As shown in Fig. 3, the "0" delays disintegrated the northeast-corner columns; the remaining columns were crippled along simultaneous diagonals, which moved sequentially toward the southwest. These diagonals are connected by the heavy solid lines. The delay number refers to the relative time at which the detonator exploded, with the number "1" delay detonating 0.5 s after the 0 delay.

This diagonalization of initiation was carried into the third dimension so that a three-dimensional wedge of columns was integrated, beginning at the bottom corner toward which topping was desired. This destruction of a critical wedge is similar to removing a wedge of a tree trunk on the side to which it is desired to fall. Just as it is not necessary to disintegrate the entire trunk to fell a tree, it is only necessary to destabilize columns on, approximately, the lower one-third of the floors. In this particular case columns were disintegrated in the basement, ground, 1st, 3rd, 5th, and 7th floors.

The duration of a typical blast initiation sequence can be seen in the time histories of ground motion presented in Fig. 4 (Contreras and Leon, 1988) for two other buildings in Mexico City, where timing of the initial explosions is shown via the air blast. The upper time history in each case represents air blast and the lower three are the three orthogonal components of ground motion. The air blast time histories do not contain any impact perturbation and can be employed to determine the length of the initiation sequence. The Durango building is similar to 158 Monterrey, and its explosive sequence lasted 4 or 5 s, with some 11 separate, delayed detonations. Delay intervals or time between successive detonations are 0.5 s per delay, in an ordinary series of electric delays. Standard delay series have much greater time intervals between explosions than the millisecond delay series normally em-

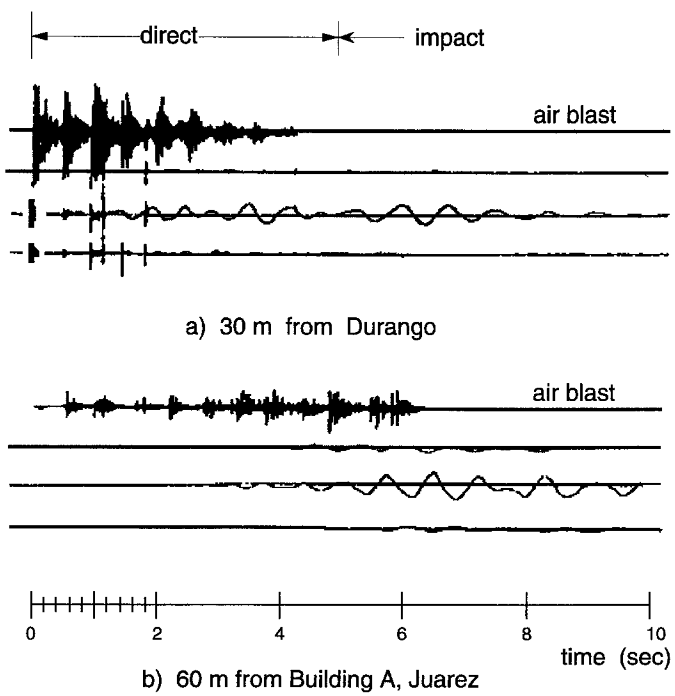

FIG. 4. Ground Motion Time Histories with Accompanying Air-Blast Overpressures

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ployed for rock excavation (Dowding 1985), where the No. 1 delay occurs 0.025 s after the No. 0 delay. These time histories are discussed in greater detail later.

While Gustafsson (1981) recommended that charges be placed along the entire height of the column, only two–four charges per column were employed to destabilize 158 Monterrey building. The larger load-bearing columns on the basement and ground floors required four and the smaller load-bearing columns of the third through seventh floors required only two. Holes were drilled to 75% of column thickness and were loaded with 240 g (0.52 lb) of Dupont Tovex. Holes were equally spaced along the full height of each lower-floor column, but located in the lower one-third of upper-floor columns. Thus, about 0.96 kg (2.10 lb) explosive were employed for each column on the lower floors, and half the amount was employed for upper-floor columns. The Gustafsson approach would have required about 1.7 kg (3.7 lb) explosive per column.

The total explosive weight detonated per delay number includes all columns detonated with that delay on all floors. About 355 charges or holes were detonated, with the largest amount detonated on the 6th initiation time or number, when 47 holes were detonated. The total explosive weight was (355 × 240 g) 85.2 kg (188 lb) and the most explosive detonated, with any one delay number, was (47 × 240 g) 11.2 kg (24.8 lb). The actual explosive weight detonated at any instant is likely to be far less than 11.2 kg because of the error in delay caps. The air-blast time histories in Fig. 4 do not show 11 single pulses, one for each delay number; rather, they show groups of peaks. Each group represents all the delay caps with the same number detonating at a distribution of times, the average of which is near the expected time. This dispersion of times has been observed by others (Dowding 1985).

DEMOLITION DETAILS FOR STEEL GIRDER BRIDGE

Use of linear shaped charges to sever steel girders has dramatically reduced the cost of demolition of steel structures. A comparison of explosive weights necessary to demolish the eight-span, 442-m-long Central Ferry Bridge in southeastern Washington illustrates this reduction ("Shaped charges" 1970). Initial estimates of conventional explosive weights of 102 kg per span raised concern for the safety of the newly constructed adjacent bridge. An alternate bid employing shaped charges was able to reduce the estimate to 1.36 kg, a reduction by a factor of almost 100.

A first step in the design of demolition blasts is the determination of the number of girder cuts needed. Over water, bridges must be cut into small enough sections to be lifted by a barge-mounted crane. Overland scrap cutting is best accomplished after the bridge is dropped in as large a piece as possible. The falling direction is chosen by designing the initiation sequence to take out rockers and supports on the side to which the tilt is desired.

The process of demolition begins with the removal of all unnecessary road surfaces, walkways, stringers, and most of the lateral bracing. Next, girders at the cut locations are preburned to reduce the length of the cut and the factor of safety to a minimum, to ensure failure upon the final explosive cutting. Fig. 1 shows the detail of the preburn and placement of the shaped charge (Flagg 1976). In this case the flanges of the stripped and unloaded bridge could be completely severed to leave only the webs for the final explosive cutting.

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TABLE 3. Structural, Blast, and Environmental Data from Case Studies

Number (1)	Name of type (2)	Weight Q (t) (3)	Area A (m²) (4)	Q/A (t/m²) (5)	Total W (kg) (6)	W delay (kg) (7)	Distance R (m) (8)	Frequency f (Hz) (9)	Particle velocity (mm/s) (10)	Air blast (dB) (11)
1	Building (4 floor)	2,000	1,600	1,3	92	9	25	26	6.00	—
								13	1.60[a]	—
2	Building (6 floor)	520	900	1.7	32	7	60	19	2.00	—
								19	0.90[a]	—
3	Building (6 floor)	1,040	900	1.7	112	27	45	9	2.40	—
								12	0.50[a]	—
4	Building (4 floor)	1,250	1,100	1.1	133	32	65	18	0.60	—
5	Building (3 floor)	200	300	0.7	22	6	18	28	11.00	—
6	Building (3 floor)	620	850	0.7	45	2	8	10	8.00	—
7	Two Building (3 floor)	1,500	2,000	0.8	168	20	30	18	6.50	—
								33	3.50[a]	—
8	Chimney	200	—	—	25	25	90	40	3.80	—
9	Building (3 floor)	1,260	1,700	0.7	300	13	8	5	8.00	—
10	Steel TV Tower	500	—	—	9	9	95	10	0.60	—
								10	0.60[a]	—
11	Building (3 floor)	350	480	0.7	35	35	175	11	0.20	—
12	Building (3 floor)	350	480	0.7	35	7	4	19	17.00	—
							50	7	1.00[a]	—
13	Thornhill Cooling Tower	3,500	110	31.8	—	—	12	—	1,524.00[a]	—
							88	—	114.30[a]	—
14	New Town, Point, London	10,000	270	4.8	—	—	42	—	6.50	—
							45	—	10.90	—

15	New Town Stratford, London	10,000	300	4.7	—	—	95	—	3.90	—
16	B. Juarez A-1	11,487	478	24.0	63	—	50	4	25.40[a]	136
							75	3	34.29[a]	133
17	B. Juarez B-1,2,3,	17,316	1,094	15.8	112	—	75	3	27.90[a]	138
							90	3	33.02[a]	134
18	B. Juarez B-4,5	11,544	729	15.8	146	—	60	3	60.96[a]	150
							180	2	25.40[a]	150
19	B. Juarez C-1	8,962	661	13.6	—	—	80	3	18.31[a]	—
							190	2	6.78[a]	123
20	B. Juarez C-2	8,962	661	13.6	—	—	95	2	45.72[a]	—
							150	2	9.14[a]	121
21	B. Juarez C-3	8,962	661	13.6	—	—	110	1	14.99[a]	130
							120	2	41.40[a]	—
22	Marina Building	8,764	768	11.4	65	—	50	2	40.60[a]	150
							60	1	35.50[a]	150
23	Monterrey no. 158	7,313	655	11.2	85	11	44	5	33.00[a]	—
24	San Antonio Abad	17,380	852	20.4	100	—	70	2	35.50[a]	150
							90	2	55.80[a]	150
25	Ministry of Labor and Public Services	15,143	1,052	14.4	125	—	60	1	50.80[a]	150
							90	2	27.80[a]	146
26	Durango no.138	12,074	670	18.0	89	—	30	2	45.70[a]	150
							75	2	43.10[a]	150
27	Jacks Run Bridge, Pittsburg, Pa.	—	—	—	634	13	45	10	58.06	>142
					634	13	45	10	45.13[a]	1

[a]Impact

CASE HISTORY DATA WITH MEASURED ENVIRONMENTAL EFFECTS

To ensure a sufficiently broad representation of projects, 130 reports and articles were collected from around the world. Despite this large number of reports, only the 27 in Table 2 were presented in a form, as of 1989, which allowed a distillation of information necessary to compare structural details, blast initiation, and environmental response. Even though the search for the measured data was quite intensive, the data base was small and derived almost entirely from sources outside the United States because most U.S. demolition companies regard any information as a trade secret. All 138 uncovered projects are compiled in an internal report (Sofrin 1988). This internal report complements previous bibliographical works on general demolition by Tibbetts (1953) and Almond (1974).

Complete project histories in Table 3 are grouped by source and type of structure, as follows:

- 01–12: older, heavy, masonry bearing wall structures from Czechoslovakia (Henrych 1979)
- 13: one of a set of cooling towers from Great Britain (Skipp et al. 1972)
- 14–15: high-rise buildings from Great Britain (New 1983)
- 16–26: medium-rise structures demolished after the September 1985 Mexican earthquake (Contreras and Leon, unpublished report, 1986)
- 27: steel truss bridge in Pennsylvania (Kitler 1987)

The parameters in Table 3 can be divided into three main groups: structural, explosive, and environmental response. Structural data in columns 1–5 include type, weight, plan area, and volume. Explosive data in columns 6–7 include total charge and, where available, charge per delay. Response data in columns 8–11 include distance to seismograph, dominant frequency, peak component particle velocity, and air-blast overpressure. Column 5, or building weight per impact area, is calculated with the building plan area in all cases except for the cooling towers. Due to its annular geometry, the impact area of a cooling tower was assumed to be one-third of the circumferential ring beam, which sustained first impact during explosive toppling.

RESULTANT AIR BLAST AND GROUND MOTION

Explosive demolition usually produces four principal external environmental effects: (1) Air blast, and/or noise; (2) ground motion; (3) projectiles; and (4) dust. Although all four are important, this paper concentrates on the first two, air blast and ground motion. Projectiles are normally prevented by the erection of barriers, which inhibit transmission of the larger but soil-sized particles, as well as dust that must be removed by the contractor. Much of the literature concentrates on projectiles and dust, which can be mitigated through temporary support and barriers as well as negotiation with adjacent property owners.

Relative times of occurrence of the air blast and ground motions can be seen in Fig. 4 for two demolitions in Mexico City. The early delays recorded 30 m from the Durango building produced ground motion as well as air blast, whereas later delays in upper stories did not couple with the ground. Disturbances produced 60 m from the demolition of Building A of the Benito Juarez housing complex show no coupling, of even delays, near the ground floor.

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Air-Blast Overpressure

Air-blast overpressure is produced by the relatively unconfined detonation of the charges in exposed girders, columns, and/or walls. These peak pressures are greater than those normally produced during typical rock blasting. As shown by the comparison of Mexico City demolition air pressures with those produced by open-air detonation in Fig. 5, air-blast levels are high. They are equivalent to 1–10 kg of explosive detonated in the open air (Siskind et al. 1980a). Air overpressures are normally given in terms of decibels (dB), which can be translated to pressure in Fig. 5, where 1 atm is 14.7 lb/sq. in. (100 kPa/m^2).

It is important that air-pressure transducers be linear, down to 2–5 Hz, to record the subaudible pressures produced by explosions. Older data acquired with transducers, which are only sensitive to the relatively high human hearing frequencies, do not record pressure transients at low structural response frequencies. Fortunately the transducers employed to monitor the blast pressures in Mexico City met U.S. Bureau of Mines specifications (Siskind et al. 1980a) and recorded the low frequency components.

Air-blast pressures produced by demolition can be large enough to break

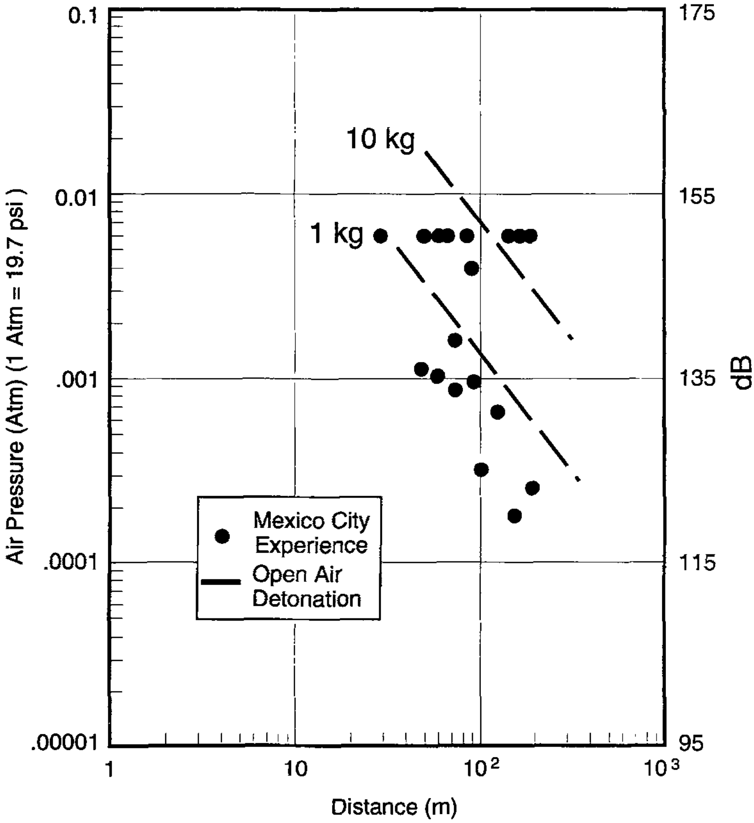

FIG. 5. Demolition-Induced Air Pressures at Increasing Stand Off Distances from Mexico City

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nearby windows; however, normal regulatory criteria should not be employed to control demolition, for a number of reasons. First, current criteria were determined for large mining shots which produce differing wave shapes. Secondly, they were established for repeated blasts, whereas demolition involves only one or two such events. Thirdly, since the blasts normally occur at one time, any breakage will be immediately obvious. Since the demolition contractor is obviously liable, repair of any broken windows and evacuation of all adjacent buildings should be prearranged before a permit is issued.

Applicable testing data for the determination of air pressures necessary to crack glass should involve detonation of small weights of explosive per delay. Windes (1943) found that glass breakage occurred above 168 dB of overpressure produced by the unconfined detonation of several pounds of dynamite at distances of 1–10 m (3–30 ft). Perkins and Jackson (1964) found that poorly mounted glass under stress could be cracked at pressure levels of 151 dB, under stress.

These test results substantiate previous reports on window damage from demolition in that only one of the 34 building demolition blasts, at the Ministry of Labor and Public Service in Mexico City, broke adjacent windows. This observation is remarkable considering that eight of these blasts produced air pressures that exceeded the 150 dB limit of blast monitors. However, a lack of reporting during a period of civilian crisis may not be reflective of the sentiment surrounding a single event in more litigious circumstances. The case involving window breakage involved the second-highest total charge weight, 125 kg.

Ground Motion

As shown by the motions 30 m from the Durango building in Fig. 4, two types of ground motion are produced during demolition. Initial, relatively high frequency motions result from the variable coupling of column removal detonations, on lower floors, with the ground; the later, much lower frequency motions are produced by the impact of the collapsing structure. The time delay reflects the time necessary for the collapse of the whole building. As shown by other time histories, motions produced by impact have greater amplitude. Thus, for the demolition of modern, reinforced–beam-column buildings in Mexico City, the most significant ground motions are produced by impact of the structure, rather than the shock of the explosive detonation itself. This ground motion pattern is observed in the demolition of steel bridge structures, in which there is no coupling with the ground during the explosive cutting of steel trusses.

The peak in the response-spectrum analysis of low-frequency ground motions, from the Durango demolition, in Fig. 6 shows the dominant frequency is approximately 1 Hz. This frequency is below the linear range for many standard blast vibration monitors, and special transducers may be needed for monitoring these low-frequency motions. Skipp and Buckley (1977) confirm the appearance of these low-frequency motions with the cooling tower demolition in Great Britain, where soils are likely to be a good deal stiffer than those in the Mexico City Valley.

As shown in Table 3, low-frequency impact particle velocities from typical beam and column structures can be as high as 55 mm/s (2.2 in./s) at 90 m (300 ft). The response spectrum of ground motions from the Durango impact in Fig. 6 shows that a 5 Hz or 2-story structure will sustain relative displacements of 0.03 in. (0.79 mm). The sinusoidal approximation of dis-

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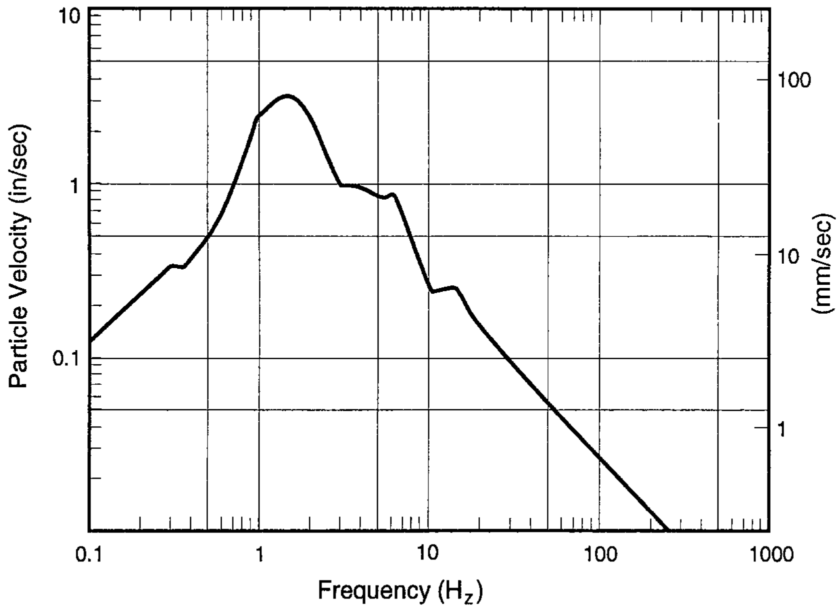

FIG. 6. Response Spectrum of Ground Motions Produced by Durango Building Impact

placement, δ, from particle velocity, v, is $\delta = v/(2\pi f)$, where f = dominant frequency. In this case where v = 25.4 mm/s (1 in./sec) at 5 Hz, $\delta = 25/(2\pi - 5) = 0.79$ mm. This displacement is large enough to induce threshold cracking in plaster walls (Dowding 1985). However, as stated in the air overpressure direction, explosive demolition is a single event where pre- and post-blast inspection and responsibility for the repair of cosmetic cracks can be negotiated with surrounding owners before the issuance of a permit.

Demolition of thick, masonry bearing wall structures, as chronicled by Henrych (1979), produces 10–30 Hz ground motions, which are dominated by initial detonation effects. In these cases, as shown in Table 3, impact motions are lower than those produced by initial detonation. These initial particle velocities are lower than those that would be produced by detonation of the explosive directly in the ground, because of the poor coupling between the aboveground structure and subsurface transmitting medium.

Fig. 7 compares impact ground motions produced by demolition of cooling towers weighing about 3500 t (Skipp and Buckley 1977), with those produced by detonation of 1 and 10 kg of dynamite confined in rock. Peak particle velocities of 10 mm/s can be produced out to 80 m by the impact of falling cooling towers. However, case history 18 [Table 3 and Fig. 8(*a*)] shows the impact of 11,500 ton buildings on soft Mexico City clays can produce ground motions that exceed 25 mm/s at 180 m.

Ground motions in the form of peak particle velocity, from the case histories presented in Table 3, are plotted in Fig. 9(*a*). As expected they decline with distance. However, velocities reported from the Mexico City cases are significantly greater than those reported by other sources. This difference may be attributable to differences in measurement systems, as

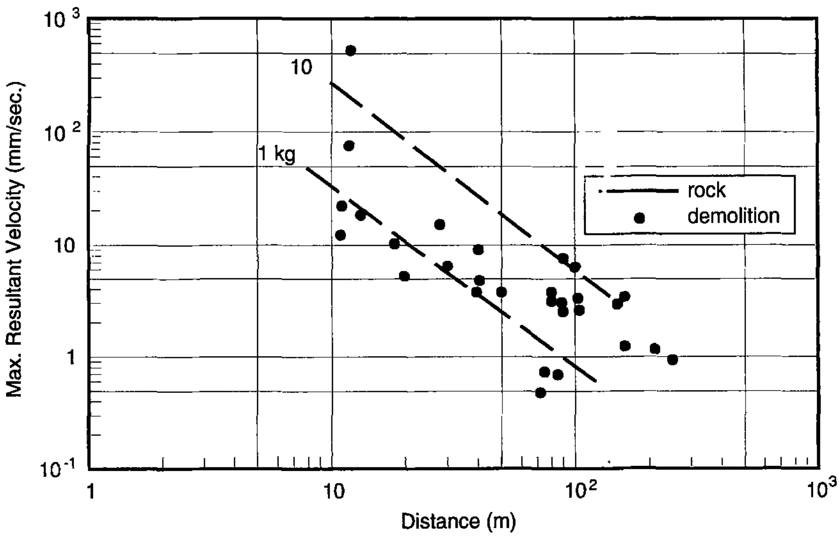

FIG. 7. Attenuation of Peak Particle Velocity by Impact of 3,500 ton Cooling Towers during Demolition

alluded to by Skipp and Buckley (1977), or may be a result of differences in impact sequence, building geometry, and/or soil conditions.

NORMALIZATION OF IMPACT MOTIONS

The peak particle velocities from the case histories in Fig. 9(*a*) have been normalized, by dividing the ratio of the building mass by the contact area, and replotted in 8(*b*). In all cases, except for the cooling towers, the contact area was assumed to be the plan area of the structure. The contact area for the cooling towers was assumed to be equal to one-third the ring beam area where initial contact is made upon impact. This normalization was successful in collapsing the data; and it could, perhaps, be collapsed further by taking into account differences of the subsoils beneath each site, except not enough soil data are available. The logic behind this normalization follows from the solution of the response of a two-degree-of-freedom system to the impact of a mass falling upon another mass.

Impact of a structure on the ground can be modeled as a one- or two-degree-of-freedom system. Such approaches have been taken to model forging hammers (Novak 1983) and pile-driving hammers (Parola 1970). Parola's model, shown in Fig. 9(*a*), will be employed in further discussion, although both of the aforementioned are applicable. The impact of the ram is analogous to that of the building, the hammer cushion spring stiffness, k, is analogous to the stiffness of the soil padding placed to cushion the building impact, drive head mass analogous to the soil cushion mass, and the pile analogous to the soil, which radiates away the energy. The solution to the resulting equations of equilibrium can be expressed in a series of dimensionless parameters; one of which (D) contains terms for the foundation soil (density, ρ, and seismic propagation velocity, c) impact area A, cushion stiffness k, and building mass m

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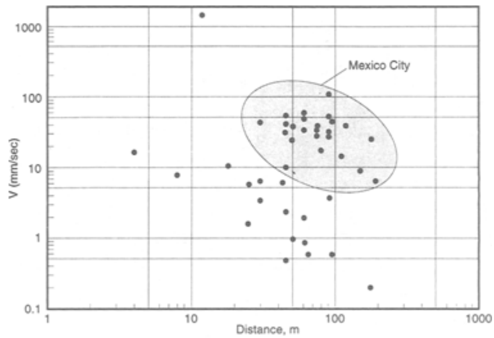
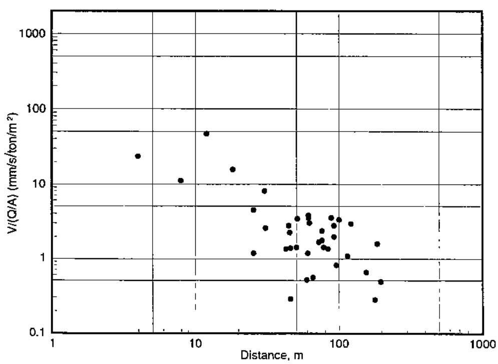


a) Particle Velocities



b) Partice Velocities Normalized by Impact Stress

FIG. 8. Comparison of Attenuation of Peak Vertical Particle Velocity from Demolition with those Normalized by Ratio of Building Weight to Impact Area

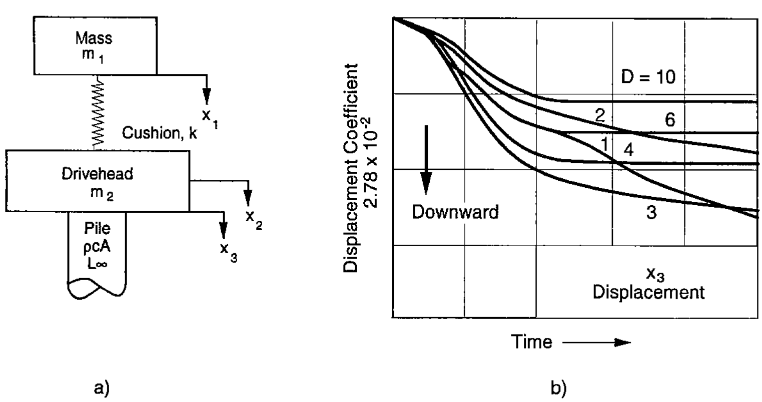


FIG. 9. (*a*) Pile Hammer Impact Analog of Building Impact; (*b*) Effect of Dimensionless Impedance Factor, D, on Pile Head Displacement

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$$D = \frac{\rho c A \beta}{\sqrt{mk}} = \frac{\rho c \beta}{\sqrt{k}\,\sqrt{m/A}} \tag{1}$$

If the ratio, β, between the falling mass of the building m_1 and soil cushion m is assumed constant, then the effect of the various parameters can be investigated.

Parola's solution for the dynamic pile-head-displacement coefficient, upon impact, as a function of D with $\beta = 5$, is shown in Fig. 9(*b*). Downward displacement is shown to increase with time during impact. If it is assumed that the ground motion measured at a distance is proportional to this downward displacement coefficient, then this relationship can be employed to estimate the effect of various parameters. The effect of parameter variation on D is calculated, which in turn modifies the expected maximum-displacement coefficient as obtained from the Fig.

First consider impact stress, or building weight, W (or mass, m) in Table 3, divided by impact area, A. As $\sqrt{m/A}$ (or $\sqrt{W/A}$) decreases, D increases, and the ultimate displacement coefficient shown in Fig. 9(*b*) decreases. Thus, ground motion should be expected to decrease as impact stresses decrease. Normalization with the initial contact stress automatically accounts for the horizontal demolition sequence because for a uniform height building, impact bay by bay will produce the same peak impact stress. However, inertial delay of upper floors may affect the impact pulse shape.

Next, consider the stiffness of the foundation soil. This stiffness is modeled by the propagation velocity c, squared times the density ρ. As soil stiffness c, increases, D increases and the ultimate-displacement coefficient decreases. Therefore, ground motion from building impact should be expected to decrease with increasing foundation soil stiffness.

Finally, consider the effect of the soil cushion normally placed beneath the building impact area. This soil is loosely dumped and should have a low stiffness k. A decrease in k increases D. In Fig. 9(*b*), increasing D again decreases the ultimate-displacement coefficient. Thus, a soil cushion absorbs energy and decreases the peak impact stress and resulting ground motion.

It appears reasonable that the particle motions recorded during the Mexico City demolition are greater than those recorded elsewhere. Since Mexico City subsoils are unusually soft, with low c's, particle velocities produced by impact may be expected to be greater, as shown by aforementioned implications of solutions to models of such impact. More data need to be collected to verify this observation.

CONCLUSIONS

Ground motions and air-blast overpressures produced by the explosive demolition of about 27 structures were compared to determine the importance of these effects. To provide background for this demolition method, general design principles were presented, along with details of the demolition of a 12-story reinforced-concrete building and a steel girder bridge. Trends in the observed ground- and air-born disturbances were compared to those expected from rock-blasting and drop-weight impact. Within the limitations of this study the following conclusions are tentatively advanced:

Since majority of data are from foreign sources, more case studies from the United States need to be reported in the literature.

Typical cases involve relatively small explosive charge weights per delay, compared with rock blasting.

Ground motions are produced by two separate sources: (1) Direct ground

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coupling (although poor) of the explosion; and (2) impact of the falling structure.

Air-blast overpressures can be atypically high compared to rock blasting, unless they are muffled.

Building impact typically produced greater motions than direct coupling of the explosions with the subsurface.

Trends in impact motions can be explained with the use of multiple degree of freedom impact models.

Regulatory guides for rock blasting may be inappropriate for explosive demolition because of the controllable singularity of the demolition event.

In addition, these conclusions point toward the following recommendations for future research: (1) More work is needed to develop exact impact models; however, more field data that include propagation velocities and stiffnesses for the impact padding and foundation soil are needed to verify the models; and (2) example specifications and/or design guidelines are needed to form a basis for permitting explosive demolition.

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