Structural Design for External Terrorist Bomb Attacks

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Events of the last ten years have greatly heightened the awareness of building owners and designers of the threat of terrorist attacks using explosives. The United States government has funded extensive research into blast analysis and protective design methods and has produced a number of guidelines for its own facilities. The private sector is increasingly considering similar measures, especially for so-called "icon buildings" that are perceived to be prime targets, as well as nearby structures that are vulnerable to collateral damage. This article summarizes the methods available to define an external terrorist bomb threat and estimate structural design loads and element responses using simple dynamic system models and principles.

Threat Definition

The details of an actual external terrorist bomb attack are by definition unpredictable. Therefore, the first design criteria that must be established for a structure that is intended to survive one are various combinations of standoff distance (R) and explosive charge size (W). R measures how close to the building a bomb could explode and is therefore a function of the physical characteristics of the surrounding site. W is expressed in weight or mass of TNT in order to correlate with tests; the equivalent W of any other explosive material is based on experimentally determined factors or the ratio of its heat of detonation to that of TNT. The effects of any blast are then normalized by the scaled distance parameter $Z = R / W^{1/3}$.

The most severe external explosive threat to a building is a moving or stationary vehicle bomb. Only the load-carrying capacity of the vehicle and the ability of the terrorist to conceal its contents limit the potential charge size. The best defense against such a weapon is to establish, as far from the structure as possible, a controlled perimeter at which every entering vehicle is inspected. Any explosive large enough to be detected during a search would then have to be detonated at that distance. This is the usual practice at many government installations, but is not always feasible for private facilities and is almost impossible in urban environments. Properly designed barriers such as bollards and special planters are another means of keeping large vehicles away from the building.

The next most critical threat is a bomb small enough to escape detection during a vehicle search or be carried and placed by hand. A terrorist could detonate such an explosive in a vehicle parked in a building's lot or garage or on an adjacent roadway, or could hide the bomb just outside the building. Parking areas located near, below, or within the structure should be secure, with access reliably limited to regular building occupants. Items such as trash containers and large equipment should not be located close to the building in such a way that they could obscure an explosive charge from view.

Finally, a terrorist may attack from a distance using direct- or indirect-fire standoff weapons such as grenades, antitank missiles, and military or improvised mortars. Although it is not practical to protect an individual building against this threat, separation from neighboring structures will reduce the potential for collateral damage from any size of explosive.

Blast Loading

Selection of the design charge size to be used for each of these conditions should not be arbitrary, but rather consistent with the attractiveness of the building itself and others nearby as terrorist targets. The designer must take into account each structure's social, economic, and patriotic significance, as well as any installed security systems and other protective measures. For example, an important building that is located well within a controlled perimeter is less likely to be attacked than it would be otherwise.

The U. S. Department of Defense, Department of State, and General Services Administration have developed specific antiterrorism requirements for military, embassy, and federal buildings, respectively. However, for security reasons key portions of these criteria are only available to designers of specific projects to which they apply. **Table 1** provides some recommendations for private-sector facilities. In all cases the designer's goal is to balance the nature and probability of each threat with the additional costs of protecting against it.

Table 1. Recommended Antiterrorism Design Criteria
(Conrath et. al.)

Tactic	Parameter	Estimated Likelihood of Terrorist Attack				Measurement of Standoff Distance R
		Low	Medium	High	Very High	
Vehicle Bomb	Vehicle Size* (lbs GVW)	4,000	4,000	5,000	12,000	Contolled Perimeter, Vehicle Barrier, or Unsecured Parking/Road
	Charge Size W (lbs TNT)	50	100	500	2,000	
Placed Bomb	Charge Size W (lbs TNT)	0	2	100	100	Unobstructed Space or Unsecured Parking/Road
Standoff Weapon	Charge Size W (lbs TNT)	2	2	50	50	Neighboring Structure

*For barrier design, with maximum velocity based on site configuration

The shock wave from an external explosion causes an almost instantaneous increase in pressure on nearby objects to a maximum value. This is followed by a brief positive phase during which the pressure decays back to its ambient value and a somewhat longer but much less intense negative phase during which the pressure reverses direction. For most structures this phenomenon can be approximated using a triangular impulse load with zero or minimal rise time and linear decay. The parameters of this equivalent load are calibrated to match the maximum reflected pressure (p_r) and total reflected impulse (i_r) of the actual load's positive phase, so that the design duration $t_d = 2i_r/p_r$. The negative phase is neglected because it usually has little effect on the maximum response.

The designer can estimate p_r and i_r for a given combination of *R* and *W* using *Z* and published curves. Although the angle of incidence at which a blast wave strikes the building surface also influences these parameters, it is usually conservative to neglect this adjustment. Either way, computer programs can perform these calculations and provide much greater accuracy. One such software product, *AT Blast*, is available for downloading free of charge from the U. S. General Services Administration (**www.oca.gsa.gov**).

Structural elements that must withstand or transfer external blast pressures--including wall and roof systems, girts and purlins, spandrel beams, columns, and frames--must be analyzed and designed accordingly. The same is true of internal elements, particularly elevated floor slabs, if windows or doors are not expected to remain intact during a blast event, since failure of these components will permit the blast pressures to propagate within the building. Although the actual blast load on an exposed element will vary over its tributary area, for design the maximum dynamic load (F_o) is typically taken as the product of this area and either the maximum pressure or a spatially averaged value. This is analogous to the manner in which design wind loads for components and cladding are routinely calculated. Blast loads need not be factored since they already represent an ultimate design condition.

Oklahoma City, OK, April 26, 1995--A scene of the devastated Murrah Building following the Oklahoma City bombing. FEMA News Photo.

Element Modeling

An element loaded by a blast can be modeled approximately as an elastic-plastic dynamic system with a single degree of freedom (SDOF) corresponding to its maximum blast deflection (y_{max}). The element's effective mass (m_e), elastic and elastic-plastic stiffnesses (k_1 and k_2), and available yield and ultimate strengths (R_y and R_u) are derived from its actual physical configuration and properties. Table 2 summarizes these parameters for a uniformly loaded one-way element of any material with various end conditions in terms of its uniformly distributed mass (m), elastic modulus (E), moment of inertia (I), span length (L), and available nominal moment capacities at mid-span (M_{nam}) and at fixed ends (M_{nae}). These values must obviously correspond to the axis of the bending induced in the element by the blast load. Similar tables exist for two-way elements with different combinations of edge conditions and aspect ratios.

When a particular element is continuously connected to an adjacent one, a portion of the latter's mass can often be added to the element's own. For example, the designer can include the mass of 20% of the wall on each side of an integral pilaster and the full tributary length of metal panels attached to girts. However, any stiffness contribution from adjacent elements should usually be neglected. When the mid-span and end moments of inertia are unequal, the designer should use the average value. The same is true of the available moment capacities at the two ends of a fixed/fixed element.

As suggested by Table 2, the simplified resistance function of a fixed/fixed or fixed/pinned element is tri-linear. Deflection consistent with the elastic stiffness k_1 occurs until initial plastic hinge formation at the yield capacity R_y, then the elastic-plastic stiffness k_2 governs up to the ultimate capacity R_u. Although it is possible to analyze the element using this resistance function, a common simplification that sacrifices little accuracy is to use an equivalent elastic stiffness calibrated to provide the same area under the curve and thus the same energy dissipation:

$$k_{1e} = \frac{k_2}{1-\left(1-\frac{k_2}{k_1}\right)\left(\frac{R_y}{R_u}\right)\left(2-\frac{R_y}{R_u}\right)} \qquad (1)$$

When the mid-span and end moment capacities are equal, this provides $k_{1e} = 307\ EI/L^3$ for fixed/fixed elements and $k_{1e} = 160\ EI/L^3$ for fixed/pinned elements.

All materials possess some inherent damping ($\xi > 0$) and some exhibit strain hardening or membrane effects ($k_3 > 0$). Such behavior is often conservatively neglected for blast analysis ($\xi = k_3 = 0$). When damping is included, it should be set at $\xi \approx 0.01$ for reinforced concrete and masonry and $\xi \approx 0.05$ for steel. The designer should use caution when assuming one or both ends pinned, since most connection details provide at least some rotational restraint and treating both ends as fixed--for stiffness calculations only--can produce a more severe dynamic response.

Table 2. Dynamic Analysis Properties of One-Way Elements
(Biggs and TM 5-855-1)

End Conditions	Mass m_e	Elastic Stiffness k_1	Ratio M_{nam}/M_{nae}	Elastic-Plastic Stiffness k_2	Yield Strength R_y	Ultimate Strenght R_u
Fixed/Fixed	0.78 m	384 EI/L^3	>1/2	76.8 EI/L^3	12 M_{nae}/L	$8(M_{nam}+M_{nae})/L$
			≤ 1/2	128 EI/L^3	24 M_{nam}/L	$8(M_{nam}+M_{nae})/L$
Fixed/Pinned	0.78 m	185 EI/L^3	> 9/16	76.8 EI/L^3	8 M_{nae}/L	$4(2M_{nam}+M_{nae})/L$
			≤ 9/16	32.8 EI/L^3	14.2 M_{nam}/L	$3.2(2.67M_{nam}+M_{nae})/L$
Fixed/Free	0.65 m	8 EI/L^3	N/A	0	2 M_{nae}/L	2 M_{nae}/L
Pinned/Pinned	0.78 m	76.8 EI/L^3	N/A	0	8 M_{nam}/L	8 M_{nam}/L

Structural Materials

Most materials used in actual construction have strengths that exceed their specified minimum values by 10% or more. In addition, the short duration of a blast load results in high strain rates that increase the design strength by at least another 10%. Consequently, for dynamic design the specified strength can be multiplied by a factor of 1.21, except that for structural steel with $F_y > 50$ ksi a factor of 1.10 is recommended. The designer can also take advantage of the increase in concrete strength with age, which for ordinary Portland cement is on the order of 10% at six months and 15% at one year or more. Material-specific interaction equations account for the reduced moment capacities available to withstand a blast load because of the stresses already present due to the dead load and a realistic portion of the live load, usually 25-50%. For dynamic analysis and design, all strength reduction or resistance factors are set to unity.

Reinforced concrete, properly detailed, is generally preferred for blast-resistant structures. Concrete masonry may also be used for exterior walls, but must always be reinforced and even then has a considerably higher potential for unacceptable brittle failure and subsequent fragmentation, especially if only cells containing reinforcing bars are grouted. Cavity walls are more effective than single wythes because the outer layer of brick will contribute additional mass and absorb many of the casing fragments produced by an external explosion.

For a reinforced concrete or masonry element, k_1 and k_2 should be based on the average of the gross and cracked section moments of inertia. An element that is subject to tension or high compression or that is slender requires a detailed investigation to determine M_{nam} and M_{nae}, and thus R_u. However, for low to moderate axial compression, it is usually conservative--albeit sometimes excessively so--to neglect the associated increase in moment capacity and use a simple interaction equation to account for biaxial bending. Applying this approach for a blast that induces x-axis bending, an element subject to moments about both axes (M_x and M_y) under dead and partial live loads and with nominal moment capacities (M_{ndx} and M_{ndy}) increased for the dynamic loading, the available nominal moment capacity (M_{nax}) is estimated as follows:

$$M_{nax} = M_{ndx}\left(1 - \frac{M_y}{M_{ndy}}\right) - M_x \qquad (2)$$

Structural steel, especially when utilized in moment-resisting frames, can tolerate a considerable amount of deflection during a blast event without collapse. However, exterior cold-formed steel wall panels or sheathed studs are often not practical for blast-resistant structures and can increase fragment hazards to building occupants. For strong-axis bending of open-section structural or cold-formed steel elements, lateral bracing of the compression flange or torsional bracing of the cross section is required at plastic hinge locations and at a spacing small enough to preclude lateral-torsional buckling. Interaction equations similar to (2), but including an axial term, must be used to determine M_{nam} and M_{nae} for a steel element subject to axial tension or compression or biaxial bending under combined dead, partial live, and blast loads.

Element Response

The designer can calculate the expected response of an element to a triangular blast impulse using published curves or a computer program capable of performing a nonlinear time-history analysis of the SDOF system. An example of the latter is *Nonlin*, available for downloading free of charge from the U. S. Federal Emergency Management Agency (**www.app1.fema.gov/EMI/nonlin.htm**). The relevant parameters for each element include the following ratios:

- Blast impulse duration to natural period of vibration ($t_d/T, T=2\pi\sqrt{m_e/k_{le}}$).
- Maximum dynamic load to available ultimate strength (F_o/R_u).
- Maximum expected deflection to yield deflection (ductility ratio $\mu = y_{max}/y_e = k_{le}y_{max}/R_u$).
- Span length to maximum expected deflection (deflection ratio $D = L/y_{max}$).

The ductility and deflection ratios correlate with the expected amount of damage to an element in a blast event, which is restricted by the level of protection that the structure must provide to its occupants and contents based on their nature, quantity, function, and importance. Table 3 describes the damage associated with various qualitative levels of protection and suggests corresponding μ_{max} and D_{min} values for one-way elements of reinforced concrete, reinforced masonry, structural steel, and cold-formed steel. The D_{min} limits are loosely based on published recommendations for the maximum end rotations (θ_{max}). For one-way elements other than cantilevers, assuming plastic hinge formation at mid-span and fixed ends and a linear deflected shape between hinges, $D_{min} = 2 / \tan\theta_{max}$. The Department of Defense has developed more detailed and less conservative response limits that are currently available only to its contractors.

As an alternative to carrying out a dynamic analysis to determine the actual μ and D for a given element and blast load, the designer can calculate the approximate limit on F_o/R_u that will ensure $\mu \le \mu_{max}$ and $y_{max} \le L/D_{min}$. Defining μ_{cr} as the lesser of μ_{max} and $k_{le}L/R_uD_{min}$:

$$\left(\frac{F_o}{R_u}\right)_{max} \approx \frac{\sqrt{2\mu_{cr}-1}}{\pi(t_d/T)} + \frac{(2\mu_{cr}-1)(t_d/T)}{2\mu_{cr}(t_d/T+0.7)} \qquad (3)$$

Table 3. Ductility and Deflection Ratio Limits for One-Way Elements
(Based on UFC 4-010-01, TM 5-1300/NAVFAC P-397/AFR 88-22, and Bounds)

Level of Protection	Description of Damage	Ductility Ratio μ_{max}				Deflection Ratio* D_{min}			
		Reinf. Conc.	Reinf. Mas.	Strl. Steel	C-F Steel	Reinf. Conc.	Reinf. Mas.	Strl. Steel	C-F Steel
High	Superficial	1	1	1	1	240	450	120	120
Medium**	Repairable	3	3	3	3	120	240	60	90
Low	Some elements unrepairable	6	6	10	3	60	150	20	60
Very Low	Overall structure unrepairable	10	10	20	6	30	120	10	30

*Multiply minimum deflection ratios by 0.5 for fixed/free (cantilever) elements
**Minimum level of protection for elements under significant axial compression

When $\mu > 1$, the element must actually be capable of undergoing the plastic deformation associated with its calculated μ and D values without suffering unacceptable damage. This requires careful detailing of members and especially connections. Although code requirements and industry guidelines for structures in high-seismic regions are helpful, they are not sufficient for blast design. Because of the localized nature of an explosion, such provisions must be followed even for elements that are not part of the lateral-force-resisting system, especially on the exterior. In case a primary supporting element does fail because of a blast, the structural system should include alternate load paths so that progressive collapse of additional bays will not follow. Multistory buildings are especially vulnerable in this respect and should have enough inherent redundancy to survive a local failure at the ground floor level.

Additional Considerations

For an element with time-varying load (F_t) and resistance (R_t), the dynamic reaction $V_t = C_R R_t + C_F F_t$, where C_R and C_F depend on the element's end conditions and ductility ratio. Since R_t and F_t reach their maximum values at different times, it is unnecessarily conservative simply to substitute R_u and F_o to obtain the maximum reaction (V_{max}). In fact, tests indicate that even the correctly calculated value of V_{max} can be several times the actual maximum shear force in an element because of the assumptions inherent in SDOF analysis, especially when the load is highly impulsive. Therefore, it is usually adequate to design the element for equivalent static values of V_{max} equal to the reactions produced by a uniformly distributed load with a total magnitude of R_u or $2F_o$, whichever is smaller. Supporting elements can then be conservatively designed to have ultimate strengths adequate to resist V_{max}.

Since the shear failure mode of concrete and masonry elements is relatively brittle, it is essential to provide appropriate reinforcement at and near supports, especially when $D < 60$ or V_{max} exceeds the capacity of the base material. The usual code equations provide the flexural shear capacity (V_c or V_m and V_s) with dynamic material properties replacing specified values. The designer must also check the direct shear capacity $V_{dc} = 0.18f'_{dc}bd$ for monolithic concrete with a given dynamic concrete strength (f'_{dc}), compression face width (b), and main reinforcement depth (d). At the face of a support where there is a construction joint or $V_{max} > V_{dc}$, adequate shear friction reinforcement is required.

Finally, the designer must account for the elastic rebound of an element subsequent to its maximum deflection, which will induce stresses opposite to those caused by the blast pressure itself. Appropriate provisions for this effect will also improve the element's ability to withstand a load reversal, which may occur if an adjacent or supporting element fails during a blast event.

Design Example 1

Given: Single-story building with eave height of 16' located at least 50' from all unsecured parking and roadways, large objects, and other structures. Likelihood of terrorist attack and required level of protection are both considered low. Exterior finish is 4" brick veneer with 2" rigid insulation supported laterally by non-load-bearing reinforced masonry walls with foundation dowels at bottom and expansion anchors at top to transfer shear only. Windows and doors are blast-resistant.

Wall Loads. Table 1: $R = 50'$, $W = 50$ lbs. $Z = 50/50^{1/3} = 13.6$ ft/lbs$^{1/3}$. *AT Blast*: $p_r = 12.6$ psi, $t_d = 7.54$ ms. Therefore, for 12"-wide portion of wall, $F_o = (12.6/1000)(12)(16)(12) = 29.0$ k/ft. Ignore compression in masonry due to self-weight.

Wall Properties: Specify $f'_m = 1{,}500$ psi, $E_m = 1{,}125$ ksi, $f_y = 60$ ksi. Estimate $f'_{dm} = 1.21(1{,}500) = 1{,}815$ psi, $f_{dy} = 1.21(60) = 72.6$ ksi. Try 8" units with centered #5 bars at 16": $A_n = 59$ in^2/ft, $I_g = 372$ in^4/ft, $I_{cr} = 45.1$ in^4/ft, $M_{nd} = 4.72$ k-ft/ft, weight is 75 psf + 40 psf brick + 3 psf insulation = 118 psf total. Neglect damping ($\xi = 0$) and treat element as pinned-pinned for both stiffness and strength, with $M_{nam} = M_{nae}$. Table 2: $m_e = 0.78(0.118)(16) = 1.47$ k/ft, $R_u = 8(4.72)/16 = 2.36$ k/ft, $k_{le} = 76.8(1{,}125)(45.1+372)/2/16^3/12^3 = 2.55$ k/in/ft. $T = 2\pi(1000)\sqrt{(1.47/386.1/2.55)} = 243$ ms.

Wall Response: $t_d/T = 7.54/243 = 0.0310$, $F_o/R_u = 29.0/2.36 = 12.3$. Table 3: $\mu_{max} = 6$ and $D_{min} = 150$. Equation (3): $(F_o/R_u)_{max} \approx 34.1 > 12.3$ (OK), $y_{max} < 6(2.36)/2.55 = 5.55$", $D > (16)(12)/5.55 = 34.6 < 150$ (NG), therefore check dynamic analysis results. *Nonlin*: $\mu = 1.26 < 6$ (OK), $D = 166 > 150$ (OK). Since $F_o/R_u > 2$, take $V_{max} = R_u/2 = 2.36/2 = 1.18$ k/ft. IBC 2000: $V_m = 4(59/1000)\sqrt{1{,}815} = 10.1$ k/ft $> V_{max}$ (OK). Shear dowel capacity $V_{dn} = (0.6)(0.31)(72.6)(12/16) = 10.1$ k/ft $> V_{max}$ (OK).

Design Example 2

Given: Same as above, except treat element as fixed-fixed for stiffness and pinned-pinned for strength. Table 2: $k_{le} = 307(1{,}125)(45.1+372)/2/16^3/12^3 = 10.2$ k/in/ft. $T = 2\mu(1000)\sqrt{(1.47/386.1/10.2)} = 121$ ms.

Wall Response: $t_d/T = 7.54/121 = 0.0623$. Equation (3): $(F_o/R_u)_{max} \approx 17.0 > 12.3$ (OK), $y_{max} < 6(2.36)/10.2 = 1.39$", $D > (16)(12)/1.39 = 138 < 150$ (NG), therefore check dynamic analysis results. *Nonlin*: $\mu = 3.51 < 6$ (OK), $D = 236 > 150$ (OK). Notice that the fixed-fixed assumption results in increased values of both μ (more severe) and D (less severe).

U.S. and Saudi military personnel survey the damage to Khobar Towers--a housing facility for U.S. service members at King Abdul Aziz Air Base near Dhahran, Saudi Arabia--caused by the explosion of a fuel truck outside the northern fence of the complex on June 25, 1996. Several buildings sustained damage, and casualties included 19 people dead and more that 500 injured. DoD photo.

Conclusion

Although it is not practical to design buildings to withstand any conceivable terrorist attack, it is possible to improve the performance of structures should one occur in the form of an external explosion. By maximizing standoff distances and hardening key elements, designers can give building occupants a reasonable chance of escaping death and serious injury during such an event. Building owners need to understand the factors that contribute to a structure's blast resistance and provide input throughout the design process to ensure that appropriate threat conditions and levels of protection are being incorporated.

Olkahoma City, OK, April 26, 1995 --The Murrah Building is demolished after the devastating explosion. FEMA News Photo.

Further Information

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A wall of the E-Ring of the Pentagon leans inward and other walls show fire damage in this Sept. 14, 2001, photograph. Damage to the Pentagon was caused when the hijacked American Airlines flight slammed into the building on Sept. 11th. The terrorist attack caused extensive damage to the west face of the building and followed similar attacks on the twin towers of the World Trade Center in New York City, DoD photo by Staff Sgt. Larry A. Simmons, U.S. Air Force.

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