STEEL FRAMED STRUCTURES SUBJECTED TO THE COMBINED EFFECTS OF BLAST AND FIRE - PART 1: STATE-OF-THE-ART REVIEW

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Abstract: Design of public infrastructure against terrorism has become a rising concern with the objective of reducing the level of damage to property and the loss of life. Some of the terrorism acts take the form of blast followed by fire causing catastrophic failure of the structure. This paper studies the response of steel framed structures subjected to the combined effects of blast and fire. A state-of-the-art summary on issues related to separate assessment of blast and fire resistance of multistory building frames is provided. This is followed by a systematic investigation on the combined effects of blast and fire using a proposed two-step dynamic analysis procedure. In Part 2 of a companion paper, a typical multi-storey building will be analyzed to study its behavior under a medium-scale explosion which triggers a post-blast fire in the affected compartment. This particular structure is found to be vulnerable as it possesses little fire resistance due to the high level of deformation caused by the blast load. The aim of this two-part paper is to advance the use of simulation tools for collapse analysis of three dimensional steel structures subject to attack by fire and explosion so that the complex interaction effects of blast and fire can be understood and quantified.

Keywords: blast, fire, Structural analysis, steel frame

1. INTRODUCTION

The main objective of designing infrastructure against terrorist acts is to reduce the level of damage to properties and loss of human life. This requires that the structure to be able to withstand local failure without disproportionate collapse. Research on assessing and enhancing civil engineering structure’s blast resistance has just begun. Guidelines and design manuals developed for military protective structures (TM5-1300, [1]) or for safety in ammunition industry (USDE, [2]) were available and found to be useful for designing civilian structures subjected to extreme loading. A series of reports was published by the Steel Construction Institute (SCI, [3]) concerning every aspect of steel offshore platforms subjected to gas explosion in which the mechanical properties of materials at high strain-rate and dynamic response of structures under impulsive loading, etc., are explained. GSA [4] provides some useful progressive collapse preventive guides for designing public buildings with special emphasis on robustness and resistance against blast loading.

The recently issued US guidelines (FEMA [5-8]) about protection of infrastructure against terrorist attacks concentrates on active protective measures such as setting of the barricades, increasing stand-off distance and providing appropriate building configuration, etc. For assessing the resistance of structural members due to blast loading, a Single Degree of Freedom (SDOF) method was firstly proposed by Biggs [9] and is later widely accepted by various design codes. With the development of computer technologies, more advanced methods have been used in research and practice. It can be expected that in the future, an advanced assessment system which involves the comparison of the blast effect to the structural resistance can be considered for the safety design of structures against blast loads.

A rational approach to fire safety assessment is to relate functional requirements, such as prevention of spreading heat and smoke, safe evacuation and rescue etc., to fire resistance considering both local and global stability of structures. In a performance-based design, the designer needs to first understand the level of performance expected, then to design to these levels and finally to predict the performance that will be achieved to ensure the reliability and robustness of the design. Nowadays, not only common constructions can be efficiently designed to meet the basic fire safety demands, but also special construction such as airport departure halls, which is unable to be designed using the conventional prescriptive based fire design code can be designed and be endowed with appropriate level of protection against fire according to the clients' requirements.

If combustible materials exist in the rooms, fire might be ignited after explosion. The interactive effect between blast and fire has been a concern in designing some of the offshore platforms and design issues are discussed by SCI (1992). However, the fire resistance of structures after the action of blast has been investigated by only a few researchers (Song et al., [11]; Izzuddin et al., [12]; Liew and Chen [13,14]). Song et al. [15] developed a program with appropriate solvers for blast and fire effects. Liew and Chen studied the resistance of some members or simple frames subjected to fire followed by blast. The major shortcoming of their work is that both the blast and fire loads are applied as nominal values and proper modeling of blast or fire loads considering their natural behaviour is not considered. Their results as an indicative value for the resistance of structural members subjected to the combined effect of blast and fire is meaningful, but gives no guidance to the real behaviour of the structure.

This paper studies the response of steel framed structures subjected to the combined effect of blast and fire. A state-of-the-art summary on issues related to separate assessment of blast and fire resistance of multistory building frames is provided. This is followed by a systematic investigation on the combined effects of blast and fire using a proposed two-step dynamic analysis procedure. Blast could damage the fire protection of the building and this would reduce the fire resistance of a building. Dynamic solution techniques to perform a sequential blast-fire analysis are discussed. In Part 2 of the companion paper, a typical multi-storey building will be analyzed for its response when subjected to a medium-scale explosion followed by fire. It will be demonstrated that structures may have strong resistance to explosions or fire alone, but possesses very little resistance to the combined action of blast and fire.

2. ANALYSIS OF STRUCTURES SUBJECTED TO BLAST LOADING

Blast load was initially considered when designing important military facilities or shelters to resist the attack of conventional or nuclear weapons. According to the investigation of DOD [16], conventional structures are designed to withstand total loads equivalent to 0.2psi (1.4kN/m^2). However, the blast overpressure imposed to the structure is from 0.9psi (6.2kN/m^2) to 12psi (82.6kN/m^2) depending on the distance and existence of barrier. A large amount of explosive attacks happen to buildings around the world every year. Designing civilian structures to be fully blast resistant is not an economically viable option. However, experience from these accidents show that huge amount of human life loss happens because of the disproportional collapse of the building. Provided that the excessive collapse of the building can be prevented, the extent of damage due to blast load could be controlled within an acceptable range. For example, the blast occurred in the basement of the WTC towers in 1993 destroyed several columns while the whole building was unaffected due to the high redundancy. The building was quickly repaired and resumed its normal function. Therefore it is possible for engineers to mitigate the effect of extreme load into a marginal level with a little increase of cost provided that appropriate structural performance analysis is performed.

2.1 Blast Phenomenon

Unconfined explosions outside of the building can be divided into two categories: air burst and surface burst. The characteristic blast wave profile from a free air burst is shown in Figure 1. The violent release of energy from a detonation gives a sudden pressure increase in the medium. The air pressure is increased instantaneously from the ambient pressure to a peak incident pressure P_{so} and then gradually diminishes to ambient pressure as the shock front travels through this specific position. The duration of the pressure is termed t_0. The shock front travels radially from the burst point with a diminishing velocity. And as the shock front expands the peak incident pressure P_{so} decreases and the duration of the pressure t_o increases. The positive overpressure is followed by a period of negative overpressure until sustained normal ambient pressure returns. The incident blast pressure at a specific position as shown in Figure 1 can be depicted by the following parameters: the incident pressure P_{so}, the incident impulse I_s, the arrival time t_a, the duration t_o, and negative phase parameters $-P_{so}$, $-I_s$, $-t_o$ in case the negative phase is concerned. These parameters are related to the charge weight W and the distance of the interested position R by empirical curves.

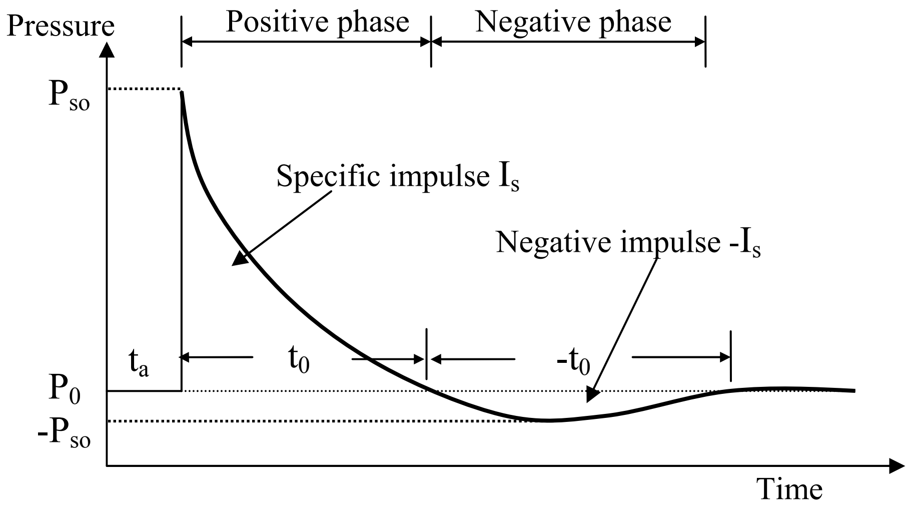

Figure 1. The variation of the overpressure at a specific position in free-air burst

After the initial blast wave or shock front has passed, a blast-induced wind, which consists of air, gases, and combustion products, causes dynamic pressures to be generated. The magnitude of the dynamic pressure Q_s is a function of the gas particle velocity and the shock front velocity. Empirically, the dynamic pressure Q_s is simply expressed as a function of the incident overpressure. Except for nuclear weapons, general explosives will be denoted near the ground surface. The initial wave of the explosion is reflected by the ground surface. The reflected wave merges with the incident wave at the point of detonation to form a single wave. Variation of the overpressure can be also depicted by Figure 1, but the blast pressure from the surface burst will be larger than that of the free air burst for the same charge weight due to the reflection effect of the ground surface. This effect can be approximately considered by enlarging the detonation weight by 1.5 - 2 times. When the shock wave impinges on a rigid surface oriented at an angle to the direction of the propagation of the wave, a reflected pressure is instantly developed on the surface and the pressure is raised to a value that exceeds the incident pressure. The pressure-time history for a reflected wave is normally assumed to be in the same shape as incident pressure in Figure 1, but with much higher magnitudes. The reflected pressure P_r and the reflected impulse I_r can be determined from the incident overpressure and the inclined angle α of the wave propagation. Calculation charts to evaluate all parameters described above can be found in TM5-1300 [1].

2.2 Classical Method for Determining Blast Loading on Structures

The classical method for determining the external blast load on buildings is derived from the concept of Mach Reflection, in which far-distance, large-scale air explosion as well as small size building prototype is assumed. In the Mach Reflection model, an air burst is assumed. The initial shock wave, when propagating away from the explosion, impinges on the ground surface prior to arrival at the structure. As the shock wave continues to propagate outward along the ground surface, a front known as the Mach front is formed by the interaction of the initial wave and the reflected wave. Along the height of the Mach front, there is little variation of the pressure (See Figure 2). For design purposes, it is simply treated as a plane wave over the full height of the front. As the Mach front propagates away from the center of the detonation, its height increases, which is referred as the path of the triple point. If the height of the triple point exceeds the height of the structure, the structure is considered to be subjected to uniform pressure. The variation of this uniform pressure with time on the side facing the detonation is shown by the time-pressure curves shown in Figure 2. At the moment the incident shock front strikes the front wall, the pressure immediately rises from zero to the reflected pressure P_r. At time t_c, the reflected wave reaches the edge of the front surface and the pressure value released to the algebraic sum of the incident pressure P_{so} and the drag pressure Q_s (see the solid curve in Figure 2). The clearing time t_c depends on the dimensions of the building and the travel speed of the reflected blast wave. However, the total impulse covered by the solid curve I_c should not exceed the reflected impulse I_r, or the reflected pressure (see the dashed curve in Figure 2) should be used as the uniform blast pressure on the building surface.

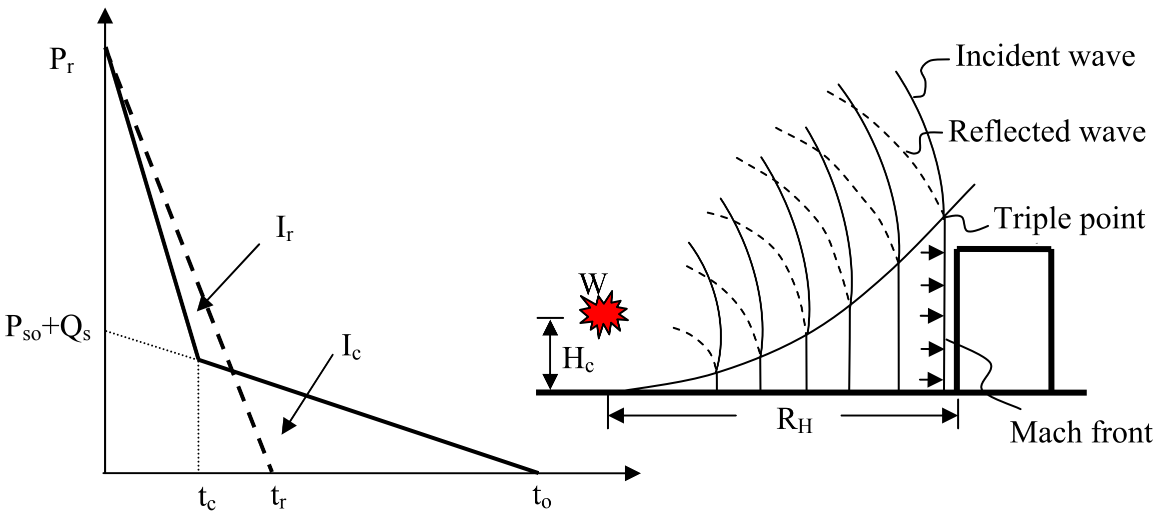

Figure 2. The Mach reflection and the variation of Mach pressure on building surface

2.3 Proposed Method to Determine Blast Load for Large Buildings

Commercial or federal buildings that are characterized to be high-rise or huge in dimension are identified to be of higher risk and a potential target for terrorist attack. The bombs, which are normally carried in cars or trucks, are detonated close to the ground and small in size compared to the building. The blast wave impinges the ground immediately after detonation and the reflected blast wave immerges with the incident wave and propagates together hemi-spherically. The blast load at a particular point on the building surface should be calculated as the reflected blast pressure at that point. TM5-1300 [1] proposes that blast pressure due to surface burst is treated as uniformly distributed over the front surface. The blast pressure at each point is a function of the distance R, inclined angle α, and charge weight W as shown in Figure. 3. It can also be written as

$$P_r = \Gamma\left(\frac{D}{W^{1/3}}, \frac{H}{D}\right) \tag{1}$$

where $\frac{D}{W^{1/3}}$ is the scaled horizontal distance with the dimension in lb and ft. Variation of the peak reflected pressure along the height of a building at various scaled horizontal distances is shown in Figure 4. $\frac{D}{W^{1/3}} = 0.60$ is equivalent to a blast of 2000 kg TNT at 3.0 m away while $\frac{D}{W^{1/3}} = 4.31$ is equivalent to a blast of 200 kg TNT at 10.0 m away. The smaller is the scaled distance, the larger is the variation of the blast pressure over the building surface. For the range of the scaled distance shown in Figure 4, it can be seen that the distribution of the peak reflected blast pressure could only be treated as approximately uniform when *H/D* is less than 0.2. Most blast attacks occurred to civilian buildings would have H/D exceeded this criterion. For example, a truck containing 2177 kg ANFO (3 tons TNT equivalent) was detonated at 3.05 m away from the front surface of the Murrah Building in the Oklahoma City in April, 1995 (Hinman and Hammond, [17]). The total height of the building is about 36 meter, with *H/D* = 11.8.

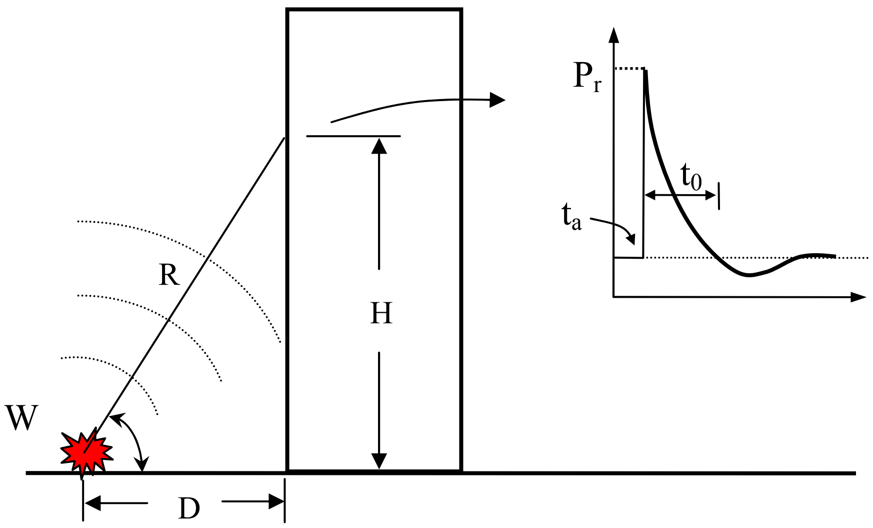

Figure 3. Calculation of the blast pressure on the building surface

Another effect that needs to be addressed is the clearance of the blast pressure due to the finite size of the building surface. It has been introduced in section 2.2 that when the blast pressure reaches the building surface, the incident pressure is reflected and accumulated along the surface. The reflected wave travels along the building surface and will be relieved when it reaches the edge of the building. If the building dimension is small, the time taken to relieve the blast pressure will be smaller than the duration of the reflected pressure duration (see Figure 2) and the actual load acting on the building surface in terms of the total impulse is smaller than the reflected impulse. Not only the edges of the building help to relieve the blast load, the presence of large openings on modern buildings such as windows or glass panels, will help to vent the blast pressure. These elements will

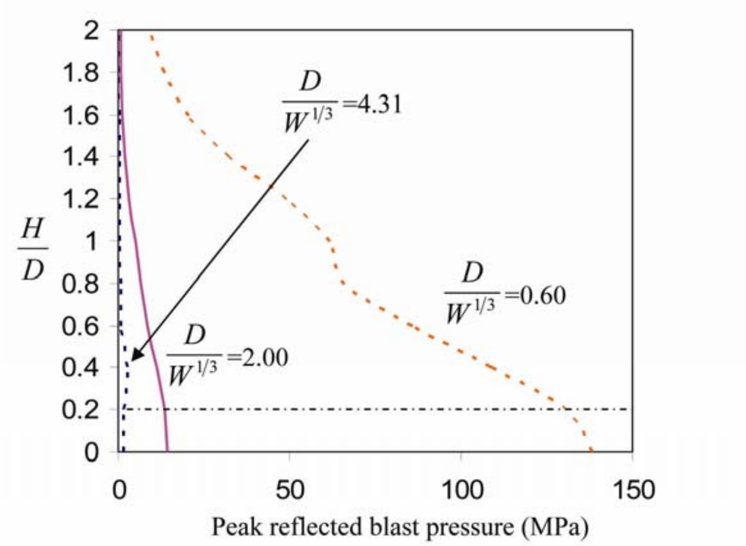

Figure 4. Variation of the peak reflected blast pressure along the building height at various scaled distances

fail almost immediately upon the arrival of the blast wave. The basic equation for calculating the clearing time is

$$t_c = \frac{4S}{(1+R)C_r} \tag{2}$$

For a rectangular panel with openings around four sides, S is the clearing distance, which is the smaller of $L/2$ or $H/2$; R is the ratio of S/G where G is the larger of $L/2$ or $H/2$ and C_r is the sound velocity in reflected region.

To compare the clearing time and the reflected pressure duration, a case study is performed as shown in Figure 5. A plate wall strip of 2 m wide is assumed to have windows both above and below it. Blast pressure can be only relieved vertically by the upper or lower windows. The clearing distance is then equal to 1 m and R approaches zero as L is assumed to be infinity. It can be seen that for this plate, the clearing effect could only become noticeable after the scaled distance exceeded 11.2 or when 200 kg of TNT is detonated at 26 m away or 1000 kg TNT at beyond 45 m. For any scaled distance less than that, the effect of openings on the calculation of the external structural loading could be simply ignored.

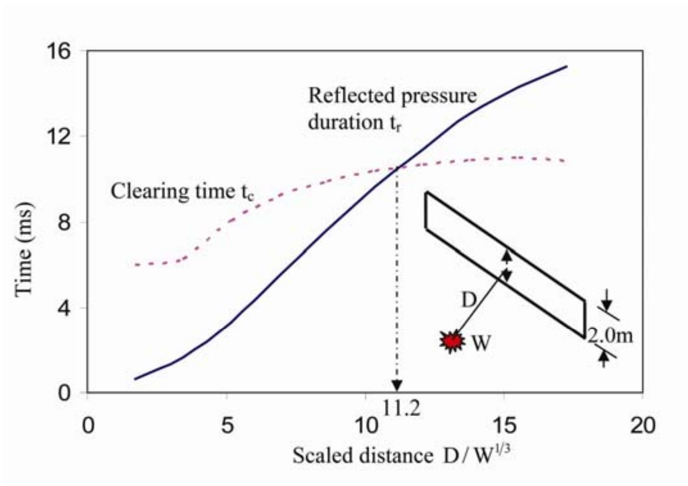

Figure 5. Comparison of the clearing time and reflected pressure duration

2.4 Analysis of Structures subjected to Blast

Blast analysis can be performed on individual members or the overall frames. The analysis methods can be varied from single-degree of freedom (SDOF)/multi-degree of freedom (MDOF) dynamic models to more complex finite element method (FEM) based on beam element model and to a highly sophisticated FEM model based on shell element model.

The SDOF method is applied to single member, which involves simplification of a continuous structural member into a dynamic system with concentrated equivalent mass and stiffness (Biggs, [9]). The responses of multi-storey frames can be represented by MDOF systems where the floors are represented by concentrated masses. Solutions to the SDOF/MDOF systems can be found by referring to the design charts or using classical dynamic solution methods. FEM (Finite Element Method) based on beam element model considers the interaction effects between individual members and the overall framework. For example, buckling or yielding of one or more members may lead to redistribution of forces and gradually weaken the structure to resist further loads. Material and geometrical nonlinearity can be included directly in the finite element formulation.

One major short coming of the beam element model is that individual member capacity is calculated based on plastic hinge mechanism. However, under rapid blast loading, the member is subjected to high shear rather than flexural deformation. Under rapid loading, the member may have failed in shear. There is also a possibility of local buckling of steel sections under blast loading. High strain rates will enhance the yield strength of materials while the Young's modulus remains almost unchanged (Liew and Chen, [15]). When the strain-rate is varied from 1.0 to $10^3 sec^{-1}$, the yield strength is increased by a factor of 1.44 to 1.79 according to the strain rate model proposed by Soroushian and Choi [18]. That means that the criterion for section slenderness is downgraded by around 20% as the section slenderness criteria are functions of $\varepsilon = \sqrt{235/f_y}$ according to EC3: Part 1.1 (CEN, 1992). The local buckling and high strain rate problems can be solved by using FEM based on shell model together with appropriate constitutive models accounting for the strain-rate effect. If more refined solid element is used, it is possible to simulate spalling and cracking of concrete or brittle failure of materials. However, refined model also means a large amount of labour in building the model and the need of intensive computation resources. Considering the inherent uncertainties in the blast loading, it should be avoided except for research purposes.

A case study is performed to compare the analysis results from the SDOF method, FEM using beam element and FEM using shell element. A steel column with section size UC356×368×129 and total length of 4.0m is assumed to be subjected to uniform lateral pressure due to blast effect. Material Grade S355 (f_y = 355N/mm^2) is used which gives the section classification for this member to be non-slender for both flanges and web. Two types of boundary conditions are considered: pin-roller ends and fixed ends. The SDOF analysis is performed according to the NORSOK (NORSOK, 2004) design code for steel members under accidental loading. The equivalent system is defined as

$$\overline{m}\ddot{y} + \overline{k}y = \bar{f}(t) \tag{3}$$

where $\overline{m} = \int_l m\varphi(x)^2 dx$, m is the mass per unit length

$\bar{f}(t) = \int_l q(t)\varphi(x)dx$, q(t) is the distribution of the load along the member

$$\overline{k} = \begin{cases} \int_l EI\varphi_{,xx}(x)^2 dx & \text{In the elastic range} \\ 0 & \text{In the plastic range} \\ \int_l N_p\varphi_{,x}(x)^2 dx & \text{In the hardening range} \end{cases}$$

$\varphi(x)$ is the shape function. It is chosen to be $\varphi(x) = \sin\frac{\pi x}{l}$ for the column with pin-roller ends and $\varphi(x) = \frac{1}{2}\left(1 - \cos\frac{2\pi x}{l}\right)$ for the column with fixed ends. There is no hardening range for the pin-roller column because it is not restrained from inward movement. For the fixed end column, N_p is the axial tension capacity of the section. The finite element analysis is performed using ABAQUS program with both beam and shell elements. The blast pressure is applied to one of the flange plate and forcing the member to bend about its major axis. Displacement about the weak axis is restrained. Geometrical initial imperfection is applied by appropriate scaling of the member's buckling shape.

Figure 6 and Figure 7 show the lateral displacement of the columns with pin-roller ends and fix-fix ends. Triangular blast loading with zero rise time is used, whose magnitudes are shown in the figures. Based on plastic-hinge theory, the stiffness of the column with pin-roller ends can be represented by a linear elastic-idealistically plastic curve. With appropriate choice of the shape function, the SDOF method gives almost exactly the same result as the FEM results using beam elements. FEM using shell elements give much larger displacement because local buckling reduces the effective stiffness of the section as shown in insert figure of Figure 6. The column with fixed ends has undergone three stages of deformation: (1) linear elastic regime, (2) inelastic transition regime where yielding occurs at the beam ends and (3) plastic regime where a plastic mechanism is formed. When an elastic fully plastic model is used (i.e., only a linear elastic regime and plastic regime is used), the stiffness tends to be over predicted. Figure 7 clearly shows that the initial stiffness predicted by the SDOF method is higher than the other two. However, the SDOF method predicts a largest maximum deflection. This is because of the error in the estimation of the stiffness in the hardening phase. For the FEM shell element model, the flanges deform toward the web upon the impingement of the blast pressure as shown in the insert figure of Figure 7. As the blast pressure is applied perpendicular to the external flange surface, the blast pressure is dissipated with the deformation of the flange. Therefore, the deflection from the shell model is smaller than the SDOF method and the FEM with beam element model.

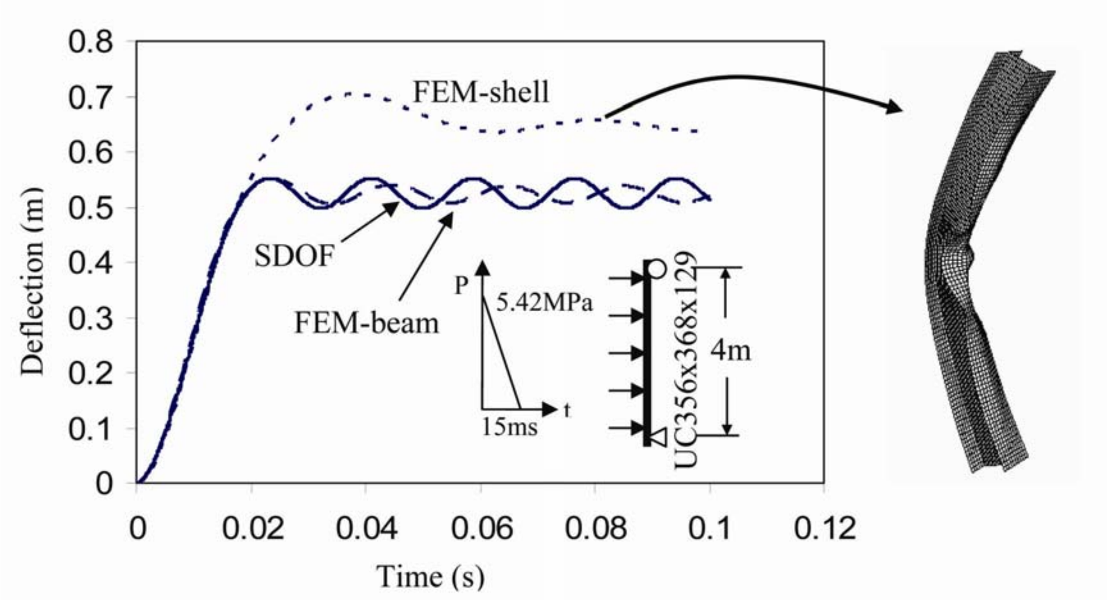

Figure 6. Comparison of the column mid-height defection with time predicted by SDOF, FEM-beam model and FEM-shell model- column with pin-roller ends

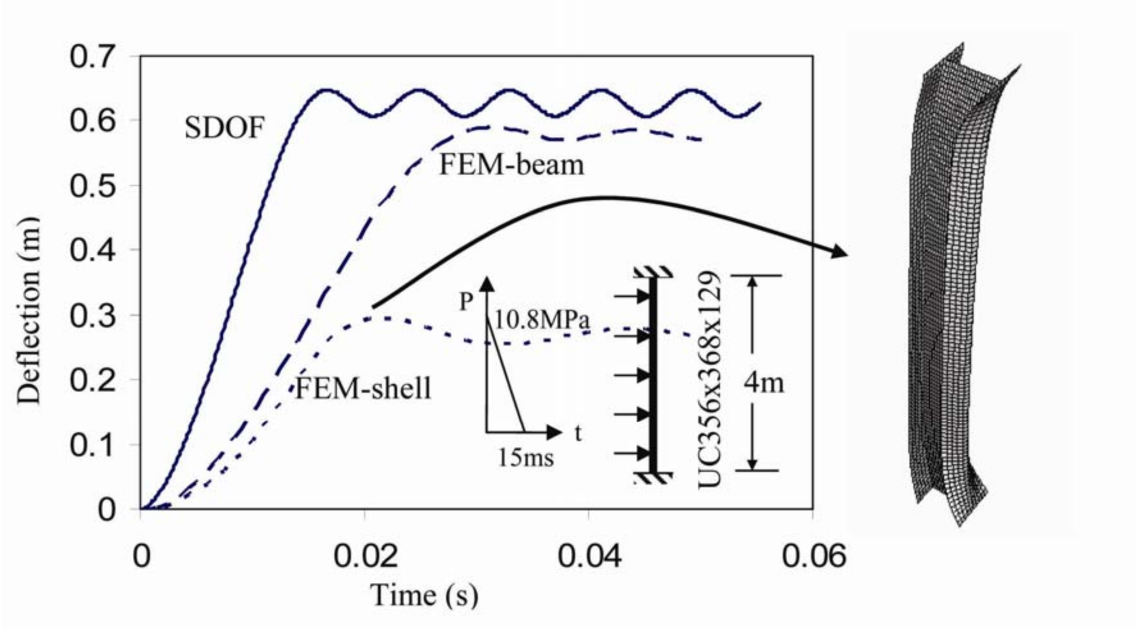

Figure 7. Comparison of the column mid-height deflection with time predicted by SDOF, FEM-beam model and FEM-shell model- column with fix-fix ends.

This example demonstrates the complexity of modeling the response of structure subjected to blast loading. The behaviour of structure depends on the difference between the blast duration and the structural natural period. The SDOF method predicts similar response behavior as the FEM-beam model provided that local deformation can be prevented and appropriate representation for the effective stiffness can be made in the SDOF method.

3. FIRE MODELING

Fire modeling is a mathematical simulation of the fire conditions in a compartment and is capable of providing information based on the parameters which have been designed. The fire development in a room normally involves three phases: pre-flashover, post-flashover and fire decay (see Figure 8). In the pre-flashover phase, fuels begin to burn and the gas temperature varies from one point to another in the compartment. In the post-flashover phase, the fire develops fully, and the gas temperature increases rapidly to a peak value and becomes practically uniform throughout the compartment. The fire has the greatest influence on structural design because of high temperature and radiant heat fluxes produced in this phase. In the fire decay phase, the available fuel begins to decrease and the gas temperature falls. There is considerable benefit when the effects of natural fires in buildings, where the amounts of the combustible contents are small and the buildings are of large volume, is considered than using the standard ISO fire.

For many years, fire engineering research has shown that the overall structure performs better than isolated members in a fire situation. Numerous studies have been carried out to determine the temperature reached in real (natural) fires, to quantify the factors that govern fire severity and to investigate the parameters that cause structures to fail in fire. The studies show that the severity of natural fires in building compartments is governed by the amount of combustible material (the fire load), the area of the doors and windows (the ventilation), and the thermal characteristics of the wall, floor and ceiling materials. In addition, fire-fighting measures are also important for the determination of fire exposure.

Eurocode 1: Part 1-2 [20,21] recommends equations for parametric fires, allowing a temperature-time curve to be produced for any combination of fuel load, opening factor, height of opening and thermal characteristic of the boundary materials (Liew and Ma, [19]). Provided that the burning parameters can be determined accurately, the parametric fire model is able to provide a good estimation of the gas temperature in a fire compartment.

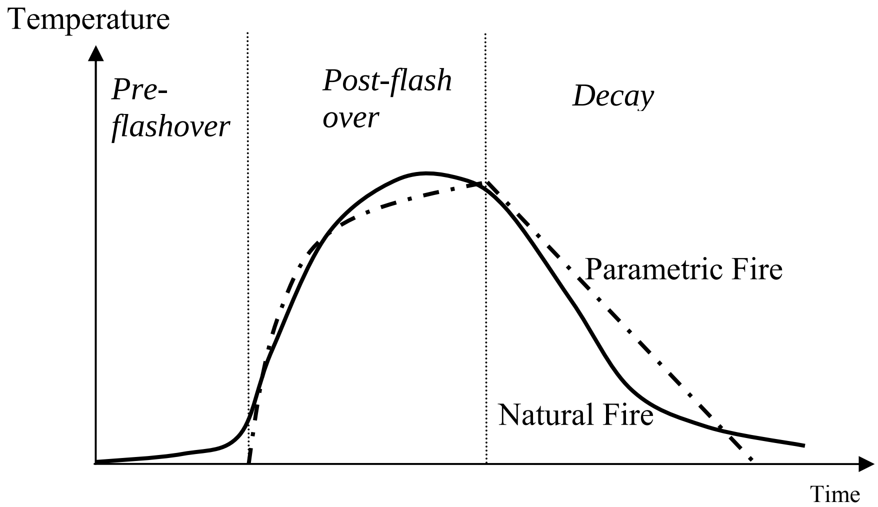

Figure 8. Comparison of the natural fire curve with a parametric fire

For a better understanding of the combustion procedure and room temperature distribution, computer programs based on CFD (Computational Fluid Dynamics) theory should be used. They work by dividing the region of interest into a large number of cells or control volumes (the mesh or grid). In each of these cells, the partial differential equations describing the fluid flow are rewritten as algebraic equations that relate the pressure, velocity, temperature, to the values in the neighboring cells. These equations are then solved numerically yielding a complete picture of the flow down to the resolution of the grid. The significant advantages of field model is ability to predict the spread of fire and smoke in compartments as well as modeling turbulence generation and dissipation, soot formation and combustion chemical reactions. Today, with the ever-increasing computing capabilities, field models have received increasing attention and spurred a lot of research and code development.

3.2 Analysis of Structures Subjected to Fire

The process of heat transfer between a fire and a structure can be described by the balance between the net incident thermal radiation and convective heat flux and the rate of heat conducted in the material. The rate of heating of any structural member is dependent at any time on the temperatures of both the fire atmosphere and the member. Calculation of member temperature requires solution in the time domain via a fairly complex differential equation.

Any analysis method that is used to predict structural response in fire must be able to take into account the change of material properties and thermal expansion of materials with increase of temperature. Simplified formulas for computing resistance of single members such as beams and columns can be found in EC3 Part 1.2 (CEN, [21]). Liew and his associates (Liew and Tang, [22]; Liew et al., [23], Liew et al., [24]) proposed the use of the plastic hinge based method to model skeleton steel structures subjected to fire. The proposed beam model can capture member stability accurately with the use of only one element per member. The beam element displacement fields are derived from the exact solution of the fourth order differential equation for a beam-column subjected to end forces (Liew et al., [22]). Material non-linearity is modeled by yield hinges at element mid-span and element ends. A more refined analysis is to use fiber elements to model the steel member in which the member cross section is divided into a number of fibers and the section forces/moments are calculated by integration of the forces at each fiber. The spread of plasticity is captured by monitoring the stress-strain relationship at each fiber. By using this model, the Euler-Bernoulli beam assumptions apply, i.e. plane sections remain plane. For a more accurate representation of detailed localized effects such as shear, warping, local deformation etc., of plate components in the member, shell element modeling is required. In case of direct analysis of the overall building response subjected to localized fires, a mixed use of different element types is required in which the fire affected members may be modeled using the shell elements and those not affected by fire may be modeled using beam elements. Some examples of the use of mixed element modeling technique can be found from Liew and Chen [14,15] and Liew et al. [25].

3.3 Protection of Buildings for Fire Safety

Fire protection to buildings can be divided into two categories: active protection and passive protection. The purpose of active protection is to decrease the possibilities of fire occurrence and fire spread. It takes the form of horizontal and vertical compartmentation, installation of fire detection system, installation of fire suppression system and availability of firefighters, etc. Passive protection is aimed to retard the load bearing members from being heated to excessive temperatures in a fire situation It generally means enclosing the structural members by thermal insulation materials such as mineral fiber or gypsum board.

The active protection measures may be taken into account in the risk and hazard factors that affect the fire development. For example, it is defined in EC1 Part 1.2 (CEN, [21]) that the design fire load should be calculated as

$$q_{f,d} = q_{f,k} m\delta_{q1} \cdot \delta_{q1} \cdot \delta_n \tag{4}$$

where $q_{f,k}$ is the characteristic fire load, m is the combustion factor, δ_{q1} and δ_{q2} are factors of the compartment size and δ_n are factors taking into account the active fire fighting measures.

Passive fire protection decreases the member temperature and is considered in the heat-transfer analysis in which the temperatures of the steel members are calculated according to the surrounding fire temperature. Simplified methods are provided by EC3 Part 1.2 (CEN, [20]) to calculate the temperature development in steel members with or without fire protection. For more complex heat transfer problems, finite element models should be built and full thermal analysis should be performed.

4. INTEGRATED BLAST AND FIRE ANALYSIS

Blast or impact is quite possibly to be followed by fire either because the detonation releases heat energy or because the blast wave damages the electricity system and causes fire. The hazard of fire to a blast or impact damaged structure is most obviously demonstrated in the collapse of WTC on Sep 11, 2001. This building was originally designed to withstand the impact of a fully-fueled Boeing 707 aircraft anywhere along the tower height. In actual case, the towers were impacted by a larger aircraft and they remained standing after the impact which damaged a large number of columns. However, it is the intense heating due to the burning of jet fuel that destroyed the vertical load bearing system and caused progressive collapse of the towers.

The crushing of an airplane to the Pentagon building on Sep 11, 2001 provided a case for more detailed study on the effect of fire to damaged buildings (Mlakar, et al., [26]). Along the outer surface of the building are concrete reinforced columns at about 3 m spacing. Impact damage on the first floor is likely to removed columns from line 10 to line 14 and the exterior columns on column lines 9, 15, 16 and 17 are severely damaged. The structure remained at the deformed shape for about 20 minutes before collapse. Investigation of the structural member's resistance to impact and fire shows that the impact damage which removed protective materials and compromised strength initially was the likely cause of the structural collapse in the following fire. The fire after impact is a typical ventilation-controlled fire whose maximum temperature is estimated to be around 800^{o}C. Thermal analysis shows that for the undamaged columns, the reinforcement needs 155 min to reach 500^{o}C when exposed to standard fire ASTM E-119, while reinforcement in damaged columns reach 500^{o}C in 50 min when subjected to the same burning because impact caused spalling of the enclosing concrete. For steel-framed structures, the impact will work similarly by destroying the fire protection layers that is generally required to enclosing the steel members.

It may be concluded from the above discussions that a stand-alone blast resistant design or fire safety design would not be adequate to meet the safety performance of such buildings with higher identified risk. A fully integrated blast and fire analysis is required.

4.1 Effect of Blasts on Fire Safety System

For a building structure subjected to a blast followed by a fire, damage to the fire safety system will impose a significant effect on the performance of the building subjected to a fire. They include:

(1) *Damage to active fire fighting system*. Active fire fighting system consists of fire detector, water supplier, and water dispenser, etc, if without proper protection, will be susceptible to damage during blast. Failure of any parts could affect the fire resistance of the building system.
(2) *Damage to passive fire protection system*: The performance of passive fire protection materials is derived from tests in which the materials are not subjected to any loads. Under blast load, some of these materials may not be properly adhered to the surface of the member. Under large displacement and yielding, the passive fire protection may fail by spalling, cracking, or de-bonding failure between material and substrate. For fire board products, opening of joints between panels may also affect their fire performance.
(3) *Damage to fire compartments*: Blast may damage the compartment walls and cause fire to spread within the building. Fire partition walls are usually designed for thermal insulation and integrity to avoid the passage of smoke and flame. As non-structural members, they have little resistance against lateral load due to blast wave or impact penetration from blast fragments.
(4) In view of global terrorism, some manufacturers have started to investigate the resistance of passive fire protections to withstand explosive fragment impacts (Pollak, 2005).

4.2 Analysis of Structures Subjected to Blast and Fire

Depending on the difference between the structure's natural period and the duration of the blast load, the structure may display three types of responses when subjected to dynamic loads. When the duration of the load is very small compared to the natural period of the structure, the structure's response is in the impulse regime. In this regime, the peak pressure is less important because the structure cannot reach the extent of deformation before the load is finished. The impulse delivers tremendous energy to the structure causing damages to the structure. When the duration of the blast load is much longer than the natural period of the structure, the loading is termed quasi-static. The quasi-static response tends to the response of an equivalent static force and a static analysis can be used to predict the response behavior of the structure. When the load duration is similar to the natural period of the structure, both peak force and the impulse will be important to determining the structural response. This is normally called dynamic regime. Dynamic analysis must be performed for structure subjected to this type of blast loading.

Methods for solving dynamic and impulsive equilibrium equations are broadly characterized as implicit or explicit. Explicit schemes obtain values for dynamic quantities at time $t+\Delta t$ based entirely on available values at time t. The central difference operator, which is the most commonly used explicit operator for stress analysis, is only conditionally stable. The benefit of explicit solver is that the governing equilibrium equation can be solved directly without the need to transpose the stiffness matrix. Convergence is ensured by definition. The main drawback is that it requires very small time steps to maintain stability, normally in 10^{-6}s to 10^{-5}s for dynamic analysis of structural members. Implicit schemes remove the constraint of using small time step by solving for the dynamic quantities at time $t+\Delta t$ based not only on values at t, but also on these same quantities at $t+\Delta t$. Because they are implicit, nonlinear equations must be solved and the nonlinear stiffness matrix needs to be transpose. For linear elastic problem, it is unconditionally stable and one time step can be used. However, for highly nonlinear problems, smaller time steps should be used to meet convergence requirements. It is generally accepted that both solvers can be used for problems falling into dynamic regime while implicit solver is more appropriate for quasi-static problems while explicit solver is most commonly used for problems involving impulsive loads.

Integrated analysis of structures subjected to blast and fire imposes a greater challenge to numerical solution technique because 1) Blast load is rapid and intense; the corresponding analysis method is characterized by the inertia effect and strain-rate effect. The material model should include strain-rate effect and preferably material failure criterion as material fracture or connection failure is often expected. 2) Fire is a relatively milder and prolonged process. While thermal creep effect is normally implicitly considered in the constitutive model, nonlinear static analysis would be applicable. The constitutive model must be able to consider degradation in the Young's modulus and effective yielding stress at elevated temperatures as well as thermal expansions.

The first attempt to perform an integrated blast and fire analysis is by Song et al. [11] and Izzuddin et al. [12]. An integrated analysis environment is developed, in which, the blast loading is solved by explicit dynamic analysis techniques and the fire loading is solved by implicit dynamic analysis. Liew and Chen [15] investigated the interaction of blast and fire load on the performance of single members and two-dimensional steel frames using ABAQUS implicit dynamic solver. Recently, Liew and Chen [14] extend the work to three dimensional steel frames in which local and lateral torsional buckling of beams was modeled and progressive collapse of building occurred when critical columns in the building buckled under the fire attack. In all these works, realistic blast and fire loadings are not considered. Instead, a simple blast loading model with variable peak pressure value is used to apply a uniformly distributed load to the member surface. A monotonic increase of the temperature with assumed temperature gradient is used for the fire analysis. The study only restricted to simple steel frames. These examples are meant to demonstrate the solution techniques, the numerical results do not usually represent the real structural behaviour.

4.3 Effect of Blast on Fire Resistance of Steel Members

An integrated analysis considering explosion followed by a fire is a very time consuming process because the analysis must be carried out in the time domain. For practical design, it would be useful to develop design charts to relate the effect of blast and fire to the resistance of individual member. Previous studies by Liew and Chen [14] produced the interaction curve for a column with UC254×254×89 section size and length varying from 3, 4.5 to 6 meters. Triangular shaped dynamic loading is used with fixed rising and decay duration. The only parameter studied is the peak load. The interaction curve is expressed in terms of the normalized peak loading P/P_{max} versus normalized fire resistance T/T_{max} where P_{max} and T_{max} are the separate blast and fire resistances of the member. The analysis results show that there is no decrease of fire resistance when the blast pressure is increased up to $0.7P_{max}$. When the blast loading is in the range of 0.9 P_{max} to $1.0P_{max}$, the fire resistance dropped rapidly from around $0.8T_{max}$ to zero.

The difficulty of generating this type of design chart is the definition of the blast resistance P_{max}. Unlike members subjected to static load, the dynamic resistance is not determined from the yield stress, but depends on the amount of energy the member can absorb. Besides, the response of a member subjected to blast is not only a function of the peak loading, but also the loading history. In the impulse regime, the impulse value determines the response while in the dynamic regime, both the peak load and the load duration determines the response of the member. When the blast loading is changed or another member is used, new design chart must be generated.

A more universally used criterion to assess the level of member damage in blast loading is the plastic utilization factor which, for a SDOF system, is defined as the ratio of the maximum deflection to the elastic deflection, or

$$\mu = \frac{y_{\max}}{y_{el}} \tag{5}$$

This parameter is independent of the blast loading and it can be conveniently calculated using the classical SDOF method.

Figure 9 shows the interaction relationship of a column subjected to blast followed by fire. The column size is UC203×203×60 and the total length is 4.0 m. The column is initially loaded with axial load equal to $0.3Af_y$. Three types of lateral blast loading are applied as shown in the figure. t/T=0.11 means the loading falls into the impulse loading regime and the maximum response depends on the impulse, instead of the peak loading. t/T = 0.45 and t/T = 1.0 fall into the dynamic loading regime where both the peak loading and the load duration are important. Figure 9 shows the interaction curves of fire resistance reduction factor (T/T_{max}) against the plastic utilization factor for various values of t/T ratios. The results suggested that an unified interaction curve, which is independent from t/T ratio, can be used for designing steel members subjected to blast and fire. It is further proposed that when the plastic utilization factor is increased from 1 to 8, the fire resistance reduction ratio decreases linearly from 1.0 to 0.7. After that, with further increase of the plastic utilization factor, the fire resistance reduction factor drops rapidly to zero.

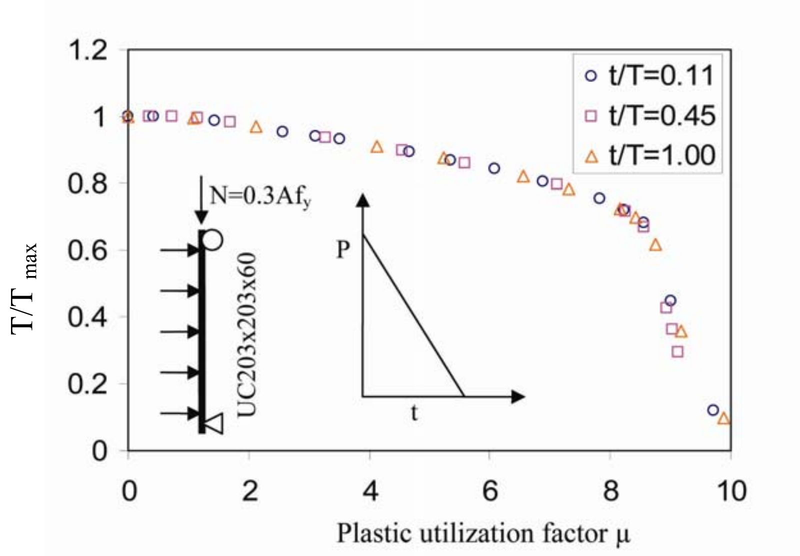

Figure 9. Interaction diagram for steel members subjected to combined blast and fire

5. CONCLUSION

This paper provides a state-of-the-art review on critical issues related to design and modeling of steel structures subjected to blast and fire. The conventional method for designing small-sized protective structures to resist the blast loading is introduced and a modified method for estimating the effect from small-sized bombs on public buildings is discussed. When dynamic analysis is performed to obtain the structural response in blast, caution needs to be taken about the potential assumptions especially when the simplified method is used. A comparison of three analysis methods shows the importance of local deformation caused by blast load and the need of more refined finite modeling technique.

Fire safety design for new buildings involves suppression of the fire and enclosing the load bearing members by heat retardant materials. These protective measures might be fully or partially damaged by the blast loads. The consequence of damage to fire protection measures is highlighted. Performing an integrated assessment of the structural resistance to blast and fire requires the structure to be analyzed in blast loading and then damaged structure to be analyzed for fire resistance. Techniques for performing a sequential blast-fire analysis are discussed. Analysis on column's response subjected to blast and fire suggests that the fire resistance of steel members will not be seriously damaged until the total displacement due to blast load is 8 times the elastic displacement. Based on the parametric study, an interaction diagram can be generated for any steel members in the form of plastic utilization factor due to blast load versus the fire resistance reduction factor for the purpose of design implementation.

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