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PERFORMANCE OF SEISMICALLY DESIGNED BUILDINGS UNDER BLAST LOADING*

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Abstract– Structures, seldom designed with adequate safety to blast hazards, are often designed for earthquake. At this backdrop, presented herein is an attempt to achieve safety against explosive hazard through aseismic design methodology. The possibility of blast damage to 'solitary' structures appears to be comparable to that due to earthquake beyond a critical standoff distance of explosion. The study develops an equivalency of blast parameters (charge weight, critical stand-off distance) and earthquake characteristics (PGA) along with ductility capacity (represented by R) so as to yield similar damage. For example, beyond a critical stand-off distance of about 25m, structures designed elastically for earthquake with PGA of 0.2g may endure little damage (similar to that at R = 2 during earthquake) due to the explosion of 500 kg charge-weight. This not only helps to ascertain the level of safety of seismically designed buildings under blast, but also to decide the distance of fencing to be constructed to protect an important structure in accordance with their functionality. However, the response in an urban setting due to similar blast action may be relatively subdued. The detailed results presented in the study may be useful to prepare codified load combinations to mitigate blast hazard.

Keywords– Blast, low-rise building, seismic, stand-off, damage, state-of-the-art, urban setting

1. INTRODUCTION

Exploring response of structures under blast has become a subject of overriding importance with a rise of blast hazards such as collapse of the Alfred P. Murrah Federal Building in Oklahoma City, Oklahoma [1, 2] and the twin towers of the world trade centre. Despite the availability of some guidelines [*e.g.*, 3] for blast-resistant design, such issues are hardly accounted in routine design of low-rise buildings, though the same are often designed for seismic hazards. During a surface burst, ground motion of high peak acceleration (PGA can be on the order of 1000g, 'g', gravitational acceleration) and short duration is induced in the vicinity of the explosion. However, such blast induced ground motions do not appear to cause serious damage. Thus, seismic hazard referred herein is related to the likely damage of the structures in the event of earthquake. In this context, it deserves mention that in the physics of explosive action, though not similar to earthquake, both blast and seismic actions are design issues related to life safety in design context [4].

In this backdrop, response of low-rise buildings (one to three stories) having different aseismic capacity, is parametrically computed under blast and code-compatible synthetic ground motions. A relative picture transpired therefrom may enable engineers to ensure structural safety, at least indirectly.

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2. STATE-OF-THE-ART AND BACKGROUND OF THE PROBLEM

An explosion is a sudden release of energy primarily causing large amount of heat and extremely high overpressure in the air adjoining the explosive that leads to potential damage. General description of blast phenomenon is enumerated at length in the literature [*e.g.*, 5-16]. However, the assessment of the effects of explosives on structural systems was apparently unexplored until World War I. One of the earliest published works in this area was that of Hopkinson [17]. In view of the increased use of explosives for security and terrorist invasion, blast resistant design of some important structures has become very important. The response of the structures subjected to both uniform and localized blast has been the subject of investigation of many studies over the last few decades. Most of the works up to the mid-1980s were exclusively concerned with the behaviour of plates and beams using simplified assumptions [18, 19]. Later on, investigations [20-26] were extended to include the effect of plate stiffeners and loading conditions on the deformation and tearing mechanism more explicitly. Prediction of the response of symmetrical cross-sections such as T-beams, quadrangular plates and stiffened plates to dynamic loading is available in the literature [21-23, 27-32]. Attempt has also been made to account for the effect of temperature on material properties as the same is intuitively expected due to high temperature emanated from the blast [33].

Endeavour towards the assessment of the overall structural behaviour under certain impulse loads was started with the development of the approximate methods using spring-mass systems having single degree of freedom [9]. An approximate quasi-static solution [34-36] for impulsively loaded structures and continua idealized as equivalent single-degree-of-freedom system was developed as an early attempt. Response due to large impact and distributed impulsive loading is attempted in the literature [37, 38].

Spatial and temporal distribution of the applied blast pressure as well as the material and geometric nonlinearities can be well-captured through detailed finite element based numerical methods. Response of a typical wall and tee-stiffened panel subject to hydrocarbon explosions was conducted [39] using FEM. Similar attempts [40] are also made by other researchers. However, such analysis requires highly specialized software and very sophisticated calibration of input parameters derived from experimentation [41]. An alternative design approach involving several approximations, but of relative ease for application in routine design, is recommended by the US Department of the Army TM5-1300 [42] and used in some other studies [43]. Such design philosophy idealizes the structural element by an equivalent single degree of freedom and the response is computed using elasto-plastic response spectra developed on the basis of typical triangular loading history with time [9]. However, the actual time-variation of blast pressure may be more appropriately described by an exponential function as observed from experimental studies [31, 44, 45]. In this context, a recent investigation [46] has developed response spectra based on exponential distribution of blast pressure. Attempts for computer simulation of blast induced pressure are also made elsewhere [47-49].

Despite great sophistication in the assessment techniques, there exists relative paucity of experimental works [31, 50, 51]. Most of the available results related to the full-scale structures are derived from the observations of the structures exposed to actual explosions [52]. Full-scale blast test on a four-story building at the White sands Missile Range in New Mexico was conducted as part of a research and development contract from the Defense Threat Reduction Agency (DTRA) to investigate measures to retrofit US Embassies and other critical structures. Structural collapse of a full-scale structure, the AMIA (Israel's mutual society of Argentina) building suffering a terrorist attack in 1994 has been simulated elsewhere [53].

Thus a review of the existing literature reveals that research progress in this area is mostly directed to the development and sophistication of computational methods to investigate the behaviour of structural components or idealized structures under blast action. In this context, response of low-rise buildings - designed under typical extreme dynamic loading such as earthquake - under blast is analyzed towards achieving safety against blast through seismic design methodology.

3. IDEALIZATION OF STRUCTURE

The idealized systems considered herein have a rigid deck slab supported by three lateral load-resisting elements in each of the two principal orthogonal directions, namely along X and Y axes, as shown in Fig. 1 (plan view) and is referred to as six-element system [54-56]. These lateral load-resisting structural elements represent frames or walls having strength and stiffness in their planes only. In most low-rise buildings, office or residential in particular, lateral load-resisting elements are generally found to be uniformly distributed over the plan. To simulate such a distribution in an equivalent sense, the idealized lateral load-resisting elements located at the centre and oriented along each of the two mutually perpendicular directions are considered to have a lateral in-plane stiffness of $2k$, which is 50% of the total lateral stiffness $4k$ in each of the principal directions. The rest of the lateral stiffness is distributed equally between the two idealized lateral load-resisting elements located near the edge along each of the principal orthogonal directions, so that each of these edge elements has lateral stiffness amounting to k. Thus the period of vibration of the system remains constant in both the orthogonal directions. The distances D between two extreme lateral load-resisting elements are kept the same in both of the two orthogonal directions.

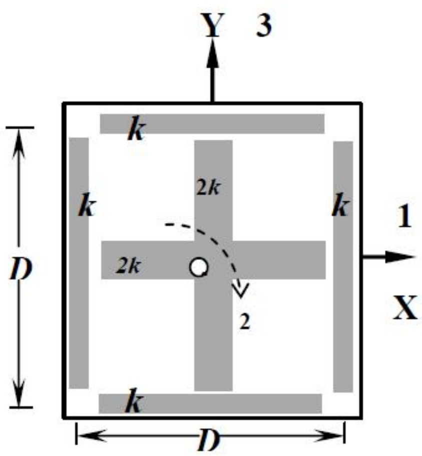


Fig. 1. Idealization of multistory building system

In the present study, the analysis has been carried out for one, two and three story buildings with fundamental lateral natural periods of 0.18s, 0.31s and 0.42s, respectively. There are six walls in the building, as shown in Fig. 1. Span of the building is 8m along both the mutually orthogonal directions and height of each story is 3.3m. Masses lumped in the floor levels are computed using the standard dimension of the structure along with the approximate unit weight of building materials.

4. IDEALIZATION OF BLAST LOAD

An explosion in air generates a pressure bulb, which grows in size at supersonic velocity. The resulting blast wave releases a large amount of energy over a small duration traveling in all directions. A typical pressure wave from an explosion consists of an overpressure phase (positive phase) and an under pressure phase (negative phase). However, for stiff systems, negative phase may not be significant relative to the positive one. Hence, the current investigation excludes the negative phase in general, as the primary objective of the present study is set to investigate the effect of blast for low-rise stiff systems. It is observed that blast-induced pressure wave decays exponentially so that the variation of the side-on overpressure may be adequately expressed as follows.

$$P(t) = P_s\left(1-\frac{t}{t_d}\right)e^{\frac{-bt}{t_d}} \quad \text{for } t < t_d \qquad \text{and} \quad P(t)=0 \quad \text{for} \quad t > t_d \tag{1}$$

where P_s is the peak side-on overpressure, t_d duration of positive phase and b is a shape parameter that dictates the manner of decay and the maximum magnitude of the negative phase pressure and is referred to as decay parameter in the rest of the study. Parameters involved in Eq. (1) for the mathematical modeling of blast load may be expressed in terms of charge weight and stand-off distance. In this context, the present study adopts widely used relationships to compute these parameters, *viz*., P_s, t_d and b relevant to a particular combination of charge weight and standoff distance. Such relationships are reproduced below from the literature [14, 57] for convenience.

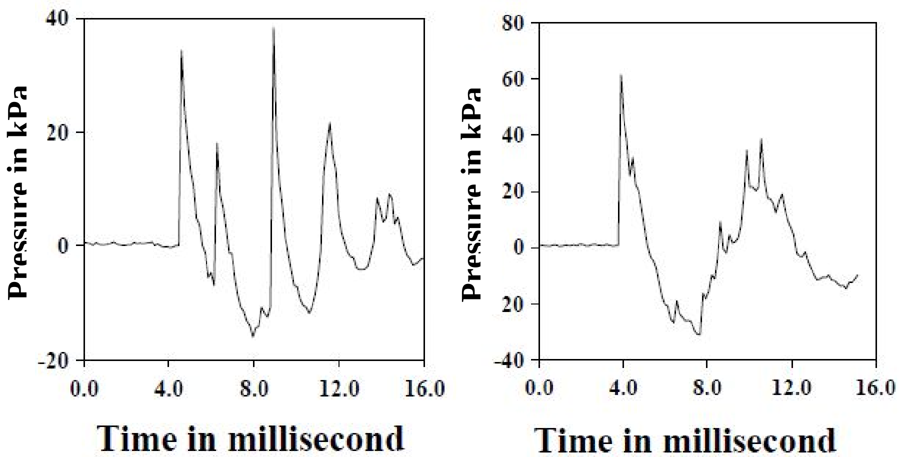
$$\frac{P_s}{P_o} = \frac{808\left[1+\left(\frac{Z}{4.5}\right)^2\right]}{\sqrt{\left[1+\left(\frac{Z}{0.048}\right)^2\right]}\sqrt{\left[1+\left(\frac{Z}{0.32}\right)^2\right]}\sqrt{\left[1+\left(\frac{Z}{1.35}\right)^2\right]}}; \quad \frac{t_d}{W^{\frac{1}{3}}} = \frac{980\left[1+\left(\frac{Z}{0.54}\right)^{10}\right]}{\sqrt{\left[1+\left(\frac{Z}{0.02}\right)^3\right]}\sqrt{\left[1+\left(\frac{Z}{0.74}\right)^6\right]}\sqrt{\left[1+\left(\frac{Z}{6.9}\right)^2\right]}}$$

$$\text{and} \qquad b = 3.18.Z^{-0.58} \tag{2}$$

where P_0 is the atmospheric pressure, Z scaled distance in m/kg$^{1/3}$ which equals $d/W^{1/3}$, d is the actual stand off distance in m and W charge weight in kg, t_d is the duration in millisecond. In practice, the positive phase of such exponential variation of pressure time history is simplified as a triangular pulse by drawing a straight line from the maximum pressure value terminating at a time t_i such that the actual impulse remains the same. The same is adopted in this paper as follows IS: 4991-1968 [3].

Further, the reflected over-pressure (P_r) arising from the interaction of the blast wave with a relatively unyielding flat surface such as the ground has been modeled by Smith and Hetherington [58] and has been conservatively modeled for zero angle of incidence as $P_r = C_r. P_s$, where C_r is the reflection coefficient available in standard literature [57, 60]. Reflected overpressure so computed has been superimposed on the static overpressure in the modeling of total impact of the blast following relevant Indian standard [3]. In this context, the 'clearing time', defined as the time taken for the reflected overpressure to decay to stagnation pressure, is estimated as per Indian standard code [3]. A recent study [61] based on realistic simulation of blast effect in urban setting indicates that the variation of reflected overpressure over the height of the building is not significant in the range of height relevant to low-rise buildings. Further, it has been shown that, for an angle of incidence exceeding 45 degrees, significantly reduced pressures and impulses are applied on the structure and, when the charge standoff exceeds one-half of the structure height (assuming the charge is centered on the structure), loads can be reasonably averaged over the structure [59]. Against this backdrop, the present investigation, assuming the entire structure in 'mach reflection region', ignores the variation of pressure as suggested elsewhere [57]. However, such consideration may cause great tolerance for higher charge weights and small stand-off distance entailing a refined modeling excluded from the scope of the present study. It may be noted that the 'mach reflection' depends on height of the burst as well as angle of reflection and, for a small height of burst along with a large distance, angle of reflection being close to 90^0, 'mach reflection' may not be observed. Thus the assumption of 'mach reflection' instead of 'regular reflection' and a consequent consideration of uniform distribution of pressure on the front wall may tend to counterweigh the effect of ignoring the impact on the side wall, rear wall and on the roof of the structure.

This idealization of the blast load appears to be applicable for 'solitary building' *i.e.*, for the buildings existing with large separation. Various government buildings, offices, historically important structures located on large premises may be treated as representative of the same. However, the distribution of the pressure and impulse generated by a blast loading in a congested urban environment is expected to be drastically different due to the interference of blast waves reflected from the nearby buildings. Systematic quantification of this effect is, however, scanty to date. Further, geometrical parameters such as width of street, height of buildings, *etc.* are known to strongly influence the same [62, 63]. In this backdrop, the present investigation considers two experimentally observed pressure-time records (refer to Fig. 2) representative of the explosive shocks occurred in urban setting, one in a relatively wide street (Fig. 2a) and the other in a narrow one (Fig. 2b) [62, 63]. Such records, corresponding to a stand-off distance of 88.0 m and a charge weight of 1000 kg TNT at full scale [62], show that the reflections coalesce and a large negative phase appears in case of explosion in a narrow street while individual reflections are distinct in the case of wide street record. Thus, it may not be warranted to ignore the impact of such negative phase and hence it is incorporated to assess the response due to airburst in congested environment, unlike solitary systems. Response of structures for similar charge weight and standoff distance is also computed using the idealization of blast load adopted for solitary system. Comparison between the response quantities so obtained may offer useful understanding to the change in response of structures arising due to the marked contrast in the nature of the blast wave in an urban setting relative to a solitary system, even for similar charge weight and explosion distance. In view of the simplified idealization of blast load employed herein, a more accurate computer simulation incorporating time-dependent effects of blast on roof, sides and the rear is needed to be made to achieve a more definitive end.



(a) (b)

Fig. 2. Nature of pressure-time variation in (a) wide street and (b) narrow street [62, 63]

5. GROUND MOTIONS

Seismic response of a structure may be sensitive to the frequency content, pattern of pulses and number of records used *etc.* In this context, seismic response of the systems, in the limited scope of the present paper, has been evaluated considering records compatible with the design spectrum of the Indian earthquake code [64], derived from that of Housner [65], as the same is also used for strength design. Two uncorrelated artificial ground motions compatible with this design spectrum are generated using a procedure outlined in the literature [66]. These synthetic earthquake acceleration histories are also used in some other studies [*e.g.*, 67]. Comparison of the target design spectrum as well as the response spectrum regenerated from these time histories, though not presented for brevity, exhibits a close resemblance. In this context, these two time histories are used in the present study to evaluate the seismic response of systems designed at various response reduction factor, *R*.

6. STRENGTH ASSIGNMENT

Elastic strength demand of the systems is first evaluated under spectrum consistent ground motion with PGA equal to 0.1g following IS: 1893-1984 [64] in the first phase of the current investigation to achieve insight into the behaviour. This is provided in the structures and considered as the elastic strength of the system with elasto-plastic hysteresis behaviour. In the process of evaluation of strength, the effect of importance factor relevant to the structure under consideration, zone in which the structure is located or the soil-foundation factor *etc.* are excluded. Subsequently, to examine the inelastic range response of systems due to seismic excitation, ground motions are suitably scaled up. The extent of inelastic behaviour of a structure with a specified lateral strength depends on the elastic strength demand due to the ground motion relative to the lateral strength of the system. Thus, the extent of inelastic behaviour of structures to be exhibited under earthquake is quantified through response reduction factor, *R*, defined as the elastic strength requirement of the system due to an earthquake as compared to the strength provided. Such parameter (*R*) is varied in the range of 1 to 6; which includes elastic to highly inelastic range behaviour as is intended for through most of the well-accepted earthquake codes [*e.g.* 68] in the world.

An increase in *R* thus indicates higher post-elastic range excursion and consequently the seismic safety of such systems relies increasingly on ductility capacity of the lateral load-resisting elements. It is, therefore, usual practice to provide higher strength to the systems having low ductility capacity and lower strength to the systems with high ductility to utilize the reserved energy absorbing capacity of members in the plastic range. In fact, as per dual design philosophy, elastic strength decides the PGA level of the moderate earthquake a structure can survive in elastic condition, while ductility capacity decides the severe earthquake (may be defined as the same moderate earthquake scaled up by the factor *R*) it will survive with acceptable damage. However, for the sake of completeness, following the PGA of design spectrum and the range of variation of the response reduction factor suggested in various seismic codes (*e.g.*, the latest version of Indian standard for earthquake-resistant design, IS: 1893-2002), systems with elastic strength corresponding to PGA equal to 0.2g and 0.4g with combinations of *R* as 1, 2, 4 and 1, 2 respectively are considered to prepare design aid for assessing the safety under blast attributed through seismic design methodology.

7. METHODOLOGY

Inertia forces acting at a story level are computed by multiplying the masses at the corresponding story level with the ground acceleration. For analysis due to blast, the product of the blast pressure and the influence area of a particular story are considered as the time-varying external force. A bilinear load-displacement characteristic is considered to represent the hysteresis behaviour of the structural elements. The non-linear equations of motion are solved in the time domain using Newmark's β-γ scheme (implicit), which considers constant average acceleration over each incremental time step. Newmark's parameters γ = 0.5 and β = 0.25 are chosen for unconditional stablility. Iterations are performed in each incremental time step with modified Newton-Raphson technique. The time step of integration is taken as less than $T_l/800$ second, where T_l is the lateral natural period of the system. 2% of critical damping in each mode of vibration is considered to constitute damping matrix.

8. RESULTS AND DISCUSSIONS

a) Influence of decay parameter ‘b’

Idealization of the blast load as depicted earlier reveals that the blast-induced pressure, its duration and the nature of variation with time as well may be expressed in terms of the charge weight and stand off distance. However, the influence of the shape of the pressure-time history as dictated by the variation of the parameter ‘*b*’ may be insignificant. This may be more clearly understood from the fact that the impact corresponding to various pressure curves obtained with a feasible range of variation of the decay parameter ‘*b*’ varies marginally. Impulse (*I*) due to overpressure-time distribution, reflective of the damaging potential of an explosive shock beyond quasi-static regime of structural response, is computed as follows.

$$I = \int_0^{t_d} P(t)dt = \int_0^{t_d} \left[P_s (1 - \frac{t}{t_d}) e^{\frac{-bt}{t_d}} \right] dt = P_s t_d \left[\frac{1}{b} - \frac{1}{b^2}(1 - e^{-b}) \right] \tag{3}$$

Impulses due to blast so computed for several charge weights exploded at various stand off distances are normalized by the same due to a rectangular pressure pulse with pressure equal to peak pressure ‘P_s’ and duration ‘t_d’. Variation of such normalized impulse has been presented in Fig. 3 as a function of corresponding decay parameter ‘*b*’ for charge weights equal to 1000 kg, 1500 kg and 2000 kg, respectively. In this context, it is worth mentioning that the parameters involved in this process such as ‘P_s’, ‘t_d’ and ‘*b*’ have been computed as per Eqs. (3a), (3b) and (3c), respectively. It is evident from Fig. 3 that the influence of decay parameter on the blast-induced impulse and hence on the damage is marginal.

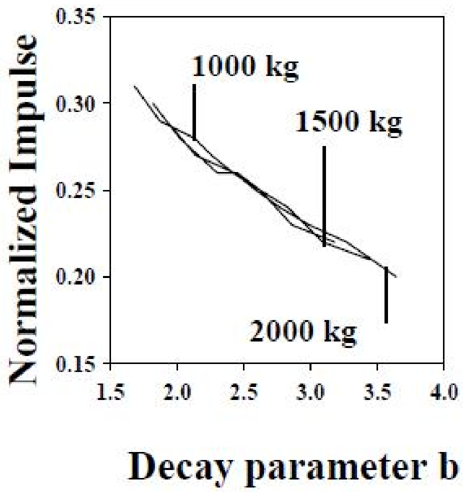


Fig. 3. Normalized impulse due to exponential overpressure-time history

-Response of low-rise buildings

1. Solitary buildings

The simplest blast-loading scenario of practical importance is perhaps the case of a solitary building (or collection of buildings with large separation distances) oriented in the direction of the blast. Response of such one, two and three story systems exhibited due to a blast generated by various charge weights expressed in terms of an equivalent amount of TNT has been computed. Such response quantity is plotted after normalizing by a similar quantity exhibited by a similar system due to synthetic ground motion. This normalized quantity, plotted against different influential parameters through Fig. 4 to Fig. 6, also indicates the ratio of the ductility demand due to blast and seismic action.

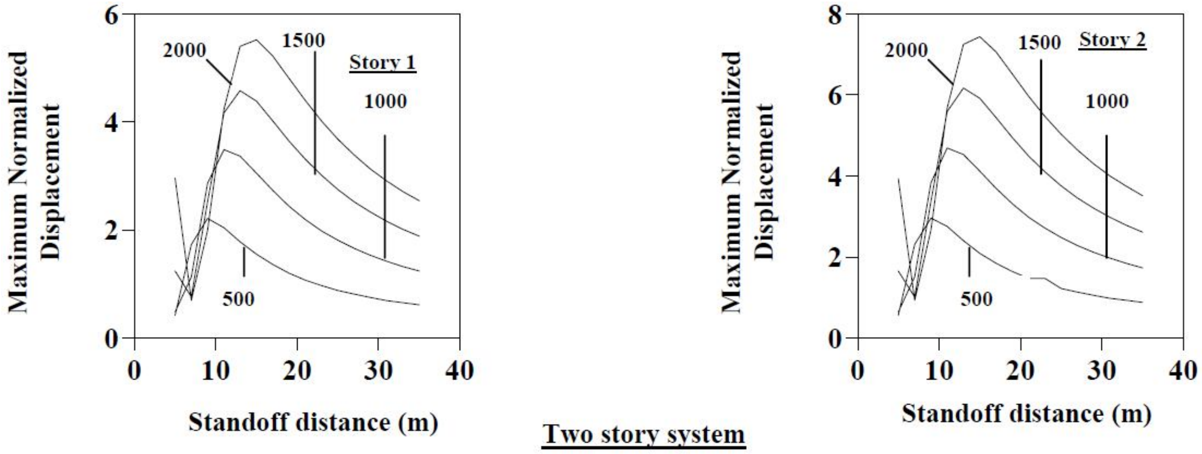


Fig. 4. Maximum normalized element displacement with variation of standoff distance (systems with R = 4; numbers shown on the curves indicate charge-weight in kg)

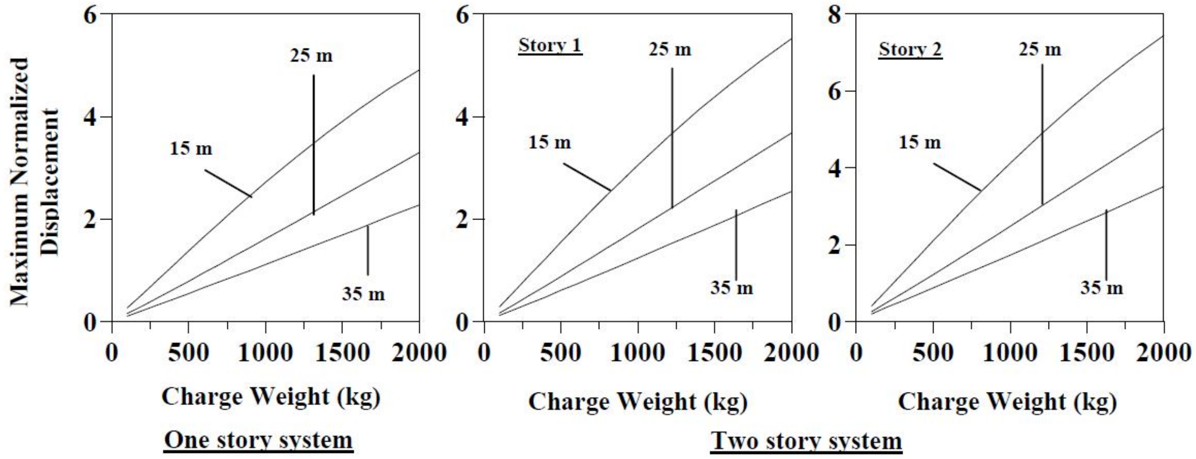


Fig. 5a. Maximum normalized element displacement with variation of charge weight (systems with R = 4; numbers shown on the curves indicate stand-off distances in meter)

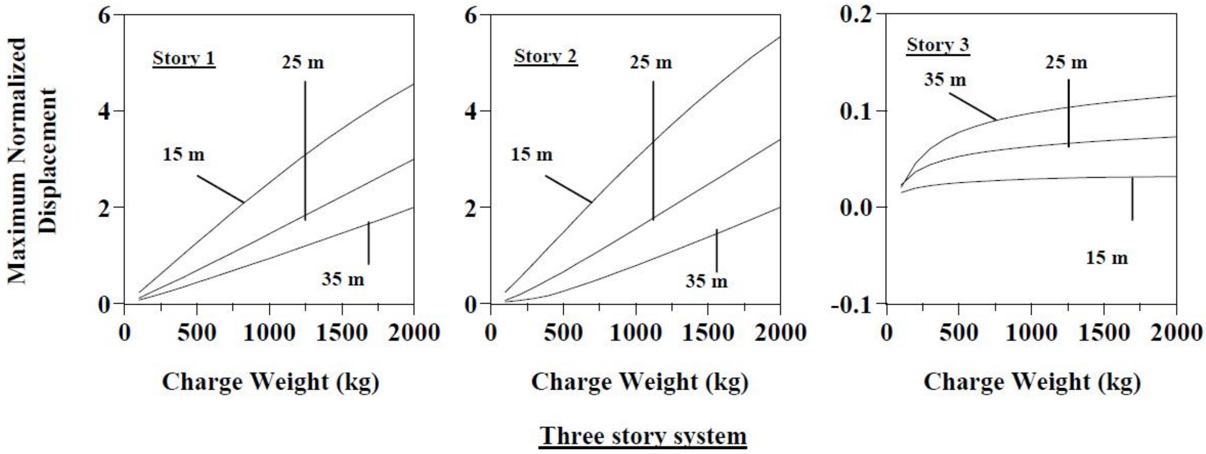


Fig. 5b. Maximum normalized element displacement with variation of charge weight (systems with R = 4; numbers shown on the curves indicate stand-off distances in meter)

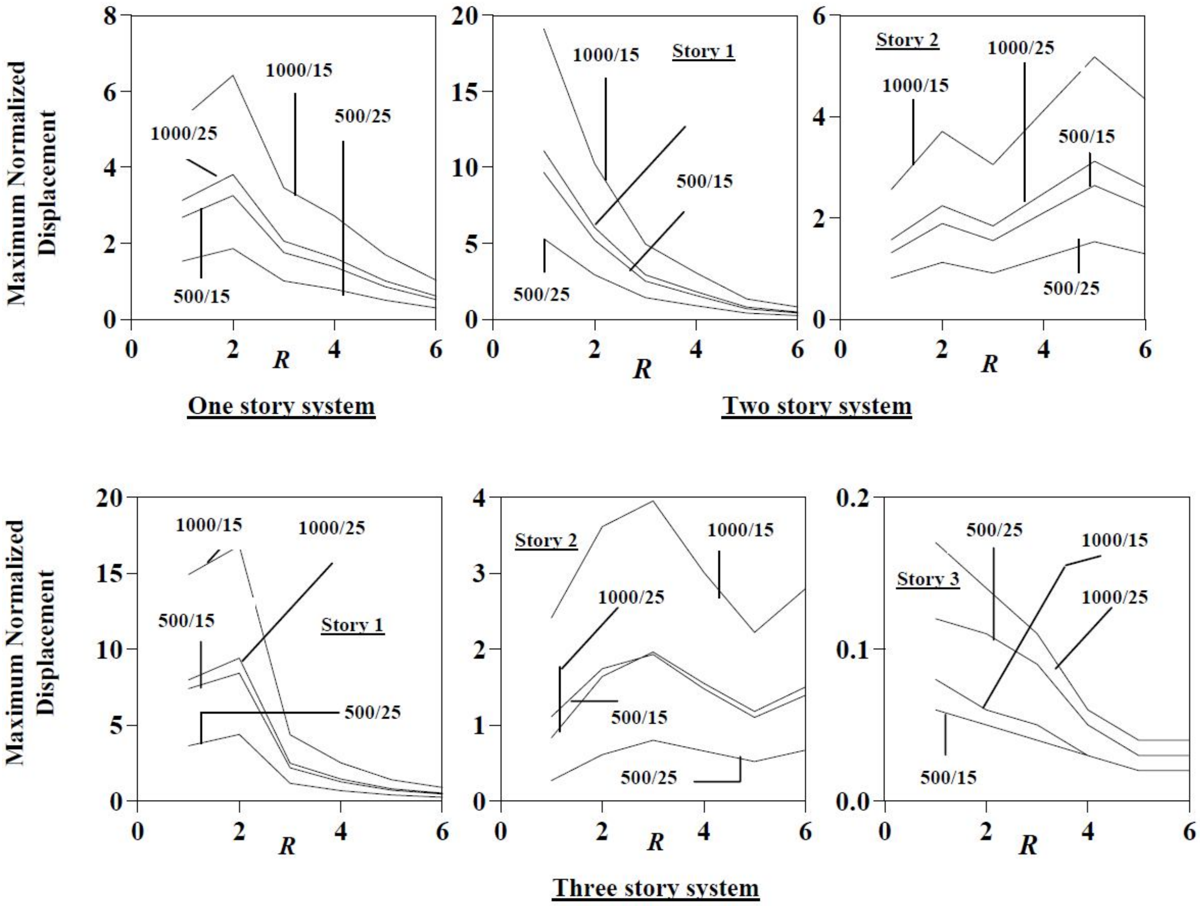


Fig. 6. Maximum normalized element displacement with variation of response reduction factor *R* [numbers shown on the curves indicate charge-weight in kg (first number) and stand-off distance in meter (second number)]

Maximum normalized element displacement is presented in Fig. 4 as a function of stand off distance for various charge weight (equivalent amount of TNT). Such response quantity has been presented for systems with *R* equal to 4 (representative of moderate inelastic exposure) in the sample form. Fig. 4 shows that such quantity attains the peak (about 4 to 7 times) for a stand off distance ranging from 10 m to 15 m with an increase in charge weight from 500 kg to 2000 kg. On the other hand, in the near field, response quantity is observed to be potentially low as compared to the same at critical distance. Intuitively, it is perceived that such trend in response should not be expected in reality. Such observation is perhaps due to the extremely complicated flow process in the near field of the explosion that limits the applicability of the empirical expressions associated with the idealization of the blast loading made in the present investigation. This points out the immediate need of further investigation of the response in the near vicinity of the explosion using rigorous modelling.

Description of the variation in response to a more closely spaced charge weight presented in Fig. 5 is expected to supplement the trend. It depicts the maximum normalized element displacement for systems with *R* equal to 4 with variation of charge weight corresponding to a set of stand off distances (15m, 25m and 35m). It is observed that such normalized displacement quantity almost linearly increases with an increase in charge weight. Further, the rate of such increase expectedly increases with decreasing stand off distance. Results show that the maximum normalized element displacement may be about 7 for load-resisting elements of the upper story in a two story system corresponding to a 2000 kg charge weight exploded at a stand off distance of 15 m.

Variation of similar response quantity is presented in Fig. 6 as a function of response reduction factor (*R*) for various combinations of charge weight and stand off distance. This shows that the response quantity generally decreases with a increase in *R*, indicative of the ductility capacity of the system. Such observation is physically intuitive. This is due to the fact that an increase in *R* implies a more severe seismic action in inelastic range. Thus, the response under seismic and blast action tends to be comparable. Variation in response also shows that the increase in response quantity is consistently lower for larger standoff distance and higher for higher charge weight. Such observation is in line with the trend exhibited through Fig. 4 and Fig. 5.

On close scrutiny of the results presented in Fig. 4 and Fig. 5, it is evident that the response attains its maximum value for a stand-off distance of about 10 - 15 m and the same reduced by about 50% at a stand-off distance of 25 m. Fig. 6 further confirms that such observation holds fairly well, irrespective of the value of the response reduction factor quantifying the level of inelasticity due to earthquake.

Overall, a critical analysis of the results reveals that the inelastic displacement response significantly increases due to blast, the duration of which is substantially small in comparison to that of ground motion. Nature of the variation of blast-induced loading appears to be similar to that due to near-fault ground motion. Seismic demand due to near-fault ground motion [67] is observed to be significantly larger compared to that due to far-field motion. Likewise such situations, in blast loading also, contrary to cyclic loading during earthquake, structures experience load with very few number of cycles. This may, perhaps, drastically enhance the demands under blast-induced actions. Furthermore, in view of the variation in response quantities, it seems physically intuitive that the damage of the structures in the event of earthquake depends on the ductility capacity of the system and design strength level expressed herein in terms of *R* and elastically endurable PGA, respectively. On the other hand, such damage during blast is essentially dependent on the blast-induced overpressure and duration of the blast itself, expressible as a function of charge weight and stand-off distance. This implies that certain combinations of blast parameters may be arrived at in that the structures, designed with a stipulated seismic safety, may safely survive. An attempt has been made to address this issue in the next subsection.

Equivalency between blast and seismic hazard: Behaviour of structures in the event of a blast relative to seismic shock, as presented in the preceding section, reveals that the blast-induced hazard is dependent on many factors such as stand-off distance, charge weight *etc.* and also intuitively on design strength and ductility of the system as well. In this context, assessment of a set of possible combinations of blast parameters yielding similar performance as that during earthquake may be interesting and useful for systems with varying strength and ductility capacity expressed as elastically sustainable PGA and specified *R*. Such an attempt may reflect the ability to cater to the explosive shocks for the structures designed with a certain degree of seismic safety. To this end, strength design of lateral load-resisting structural elements is made under spectrum compatible ground motion with *PGA* of 0.2g and 0.4g following the recent version of Indian code for earthquake-resistant design [68]. Response reduction factor *R* is considered as 1, 2 and 4 for the first case; while for the latter, only 1 and 2 are considered as already mentioned. Likewise, systems designed with *PGA* equal to 0.1g as per older version of relevant Indian standard [64] is also considered for *R* of 1, 2 and 4. Subsequently, ductility demands of structural elements due to spectrum compatible ground motion are evaluated for all above-mentioned combinations and the same are assumed to be attributed to the ductility capacity of the elements.

Thus, strength and ductility capacity are quantified in terms of the ground motion characteristics, *viz.*, *PGA* and scale factor applied to such ground motion, *R* . Such systems are analyzed due to blast, with a view to arriving at feasible combinations of blast parameters satisfying the attributed strength and ductility requirement due to various levels of earthquake causing no damage (*R* = 1), little damage (*R* = 2) and severe damage (*R* = 4), respectively. Results of such analyses are presented in the form of a bar chart in

Fig. 7. It may be noted that a few cases (*e.g.*, *PGA* = 0.2g, *R* = 4; W = 500 kg and *PGA* = 0.4g, *R* = 4; W = 500 kg, 1000kg) are excluded from the comparison as the relevant stand-off distances are found to be located in the near-field where empirical expressions used herein for idealization of blast may not be applicable. Fig. 7 shows that the critical distance of explosion (critical stand-off distance), beyond which damage due to blast approaches caused by earthquake in which the structure can survive either elastically (R = 1) or inelastically ($R \geq 2$), reduces with increasing ductility capacity and design strength of the system. Thus, Fig. 7 serves as an index to decide the level of seismic design necessary to survive certain combinations of charge weight of a blast with a particular stand-off distance and hence, may help to judge the safety level of the building during airburst. For instance, a structure designed to resist earthquakes with PGA 0.2g and 0.4g, respectively, may be able to resist the blast of a 500 kg charge-weight at a stand-off distance of about 40 m and 25 m or more; and 25 m and 20 m or more, respectively with no or little damage. Similarly, for structures designed with a seismic protection against earthquakes with a PGA of 0.2g, an explosion due to 2000 kg charge weight from stand-off distances about 85 m, 60 m and 25 m or beyond may yield no, little or severe damage which are comparable respectively to the seismic response for *R* equal to 1, 2 and 4. However, structures may survive with no or little damage under a similar charge weight with comparatively lower stand-off distances of about 55 m and 40 m or more respectively, provided the systems are designed for earthquakes with a PGA equal to 0.4g. At the same time, if a similar structure is attributed with a ductility capacity as expected for *R* equals to 4 indicating the capability of the system to safely accommodate plastic deformation caused by a four times scaled up earthquake relative to that with a PGA of 0.4g, the critical stand-off distances reduce to 12 m and 14 m respectively for an explosion of 1500 kg and 2000 kg charge weights, utilizing this inelastic deformation or ductility capacity. However, intuitively it appears that such distance is too small and in reality the structure may experience serious damage. Thus, the same, likely to be located in the near-field deserves further investigation with a refined modeling of air-blast phenomenon. All such information may help to decide the minimum distance of fencing required for protecting the structures and broadly helps to assess the level of blast loading the structures can survive.

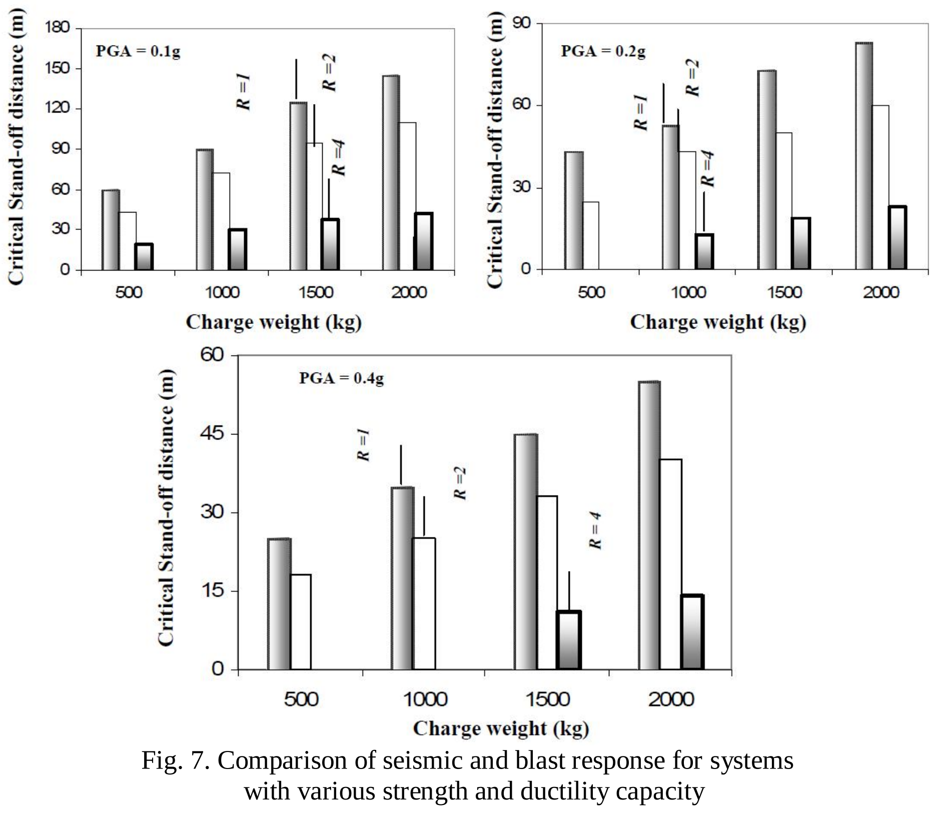


Fig. 7. Comparison of seismic and blast response for systems with various strength and ductility capacity

In this context, it is interesting to refer to a recent enlightening contribution [69] similarly comparing the blast response to the seismic response. Structural ductility (μ) of nine low-rise steel frames with different geometric parameters (*viz.*, bay length, number of story *etc.*) are evaluated therein. For each system, response reduction factor R is computed using 'nonlinear pushover blast force-displacement curves'. It is observed that 'blast related' R increases with increasing μ similar to seismic action. However, a compilation of earlier works on 'earthquake-related' R with increasing μ reveals that the 'earthquake-related' R may be potentially higher than the 'blast-related' R for specified μ. Conversely speaking, in the context of design, structures designed with a specific response reduction factor R are subject to higher ductility demand due to blast relative to earthquake.

The above observation may be interpreted in light of the inherent characteristics of these two extreme loads. Explosive load is characterized by a single high pressure impulsive pulse, while earthquake introduces vibrational energy to the system [4]. Thus, for a similar increase of μ, because of the reversible nature of earthquake unlike blast, the increase of ability to withstand inelastic range response measured by R appears to be higher during earthquake relative to blast. From a design perspective, for a specific design R, because of such reversible nature of earthquake, ductility demand appears to be low under seismic action compared to blast, and hence the influence of ductility may be less significant for blast loading. However, a consistent trend may not be traced for systems with a strength and stiffness degrading hysteresis model such as reinforced concrete.

Nevertheless, it must be remembered that 'the importance of ductility in reducing the blast loading is evident' 'irrespective of the period of vibration of the system' [4, 58, 69, 70]. Since the objective of the current undertaking is to achieve some confidence for the blast hazard through the seismic design strategy, the attempt to prepare equivalency between blast parameters (such as stand-off distance, charge weight *etc.*) and seismic design criteria (elastic strength, ductility) appears to be useful for practical purposes.

2. Buildings in urban environment

Occurrence of explosion in an urban environment leads to very different nature and magnitude of pressure-duration curves due to the reflections from the nearby buildings. Thus, the results presented in the previous section may not be applicable for similar systems in a congested urban environment. Limited attempt is, therefore, made to examine the behaviour of similar systems under pressure history containing the characteristics recorded in a typical urban environment. Response due to such pressure history normalized by the same in a solitary system due to an otherwise similar explosion (identical charge weight and stand-off distance) has been presented in Fig. 8 as a function of *R*. Response under pressure exhibiting nature expected in a typical narrow street and wide street urban environment as reported in the literature [62, 63] is included in the same figure by solid and dotted lines, respectively. Fig. 8 indicates that the increase in response reduction factor leads to a decrease in the response of lateral load-resisting elements of the lower story, while increasing in the response of those in the upper stories. Fig. 8 further shows that the demand quantities due to blast in an urban environment appears to be quite low compared to that for a solitary system. This observation is physically intuitive. The monotonic pressure-history impinged on an elasto-plastic solitary system is expected to yield a higher displacement demand as it keeps on pushing the mass in the same direction, even for identical peak-pressure and duration of the pulse. These limited results indicate that a solitary building may be used for study under blast loading as it is subjected to more potential danger relative to an urban setting. However, the potential danger in an urban environment may primarily arise due to the possibility of an explosion with a very short-stand-off distance and a simultaneous effect on many structures near the point of explosion. These issues need to be explored in future using hydrocodes [61, 62].

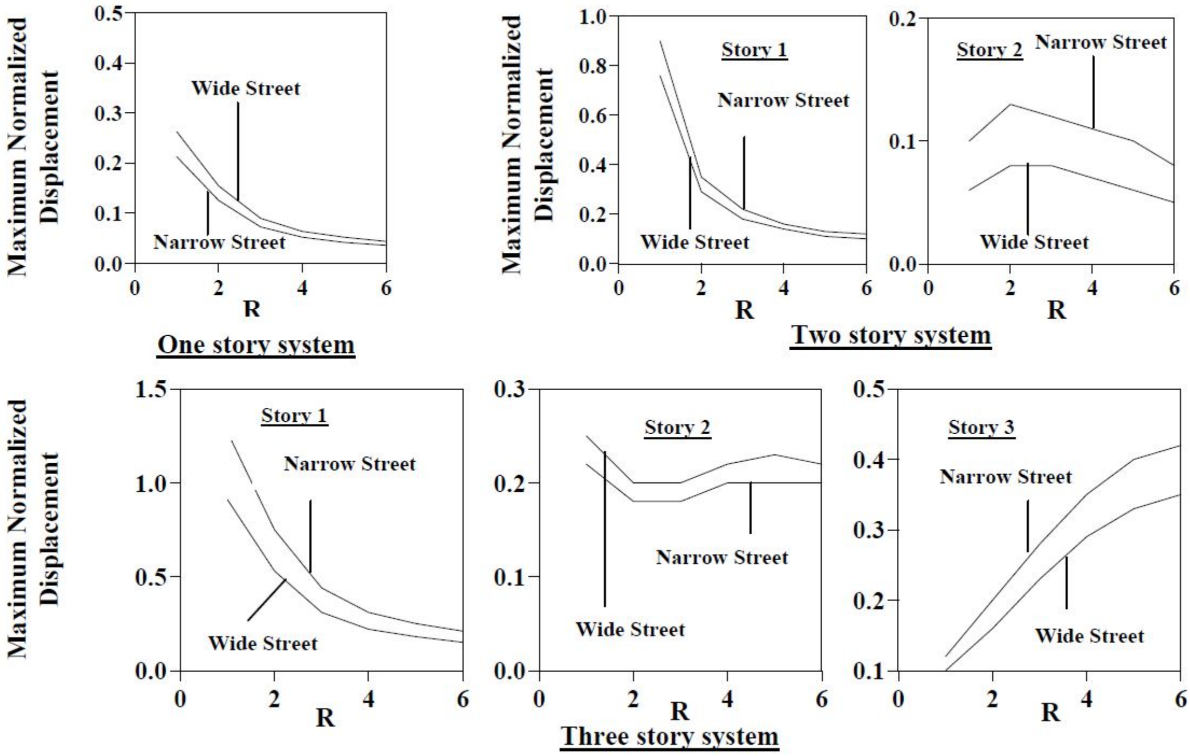


Fig. 8. Maximum element displacement due to blast in typical urban environment normalized by that of solitary system under similar blast

9. SUMMARY AND CONCLUSIONS

Design guidelines for blast load, though exist, are seldom used in the design of low-rise buildings. However, such systems are often designed with seismic protection. In this backdrop, the paper attempts to compare the performance of low-rise systems designed with seismic protection during explosion. The study may lead to the following broad conclusions.

1. Influence of blast load may cause a significant increase in the inelastic demand of the structures, even if the systems are appropriately designed for another extreme lateral load induced due to earthquake. The results presented may be indicative of displacement and ductility capacity required to survive the blast-induced hazards in relation to what is required under seismic excitation.
2. There exists a critical standoff distance beyond which damage due to the occurrence of an explosion of a specified charge-weight may become comparable as that under earthquake. Such a critical stand-off distance may vary significantly depending on the attributed strength and ductility capacity of lateral load-resisting structural elements.
3. Such critical stand-off distance of explosion for a set of charge-weights relevant to the structures designed with feasible combinations of strength and ductility capacity presented herein in the form of a bar chart may be useful to achieve the safety of structures due to blast.
4. Beyond a critical stand-off distance of about 40m and 25m, structures designed for earthquakes with a PGA of 0.2g and 0.4g, may survive the explosion of a 500 kg charge-weight with no damage. However, such critical stand-off distance may be lowered to around 25m and 20m respectively if the structures can endure a little damage through its ductility associated to a response reduction factor, R = 2. Similar distances relevant to a comparatively severe charge-weight of 2000 kg yielding no damage may be on the order of 85m and 55m for systems having strength for an earthquake with a PGA of 0.2g and 0.4g, respectively. Such quantities appear to be reduced around 60m and 40m for systems capable of

undergoing little damage. Further, for systems with ductility capacity compatible with R = 4 and elastic strength consistent with an earthquake having a PGA of 0.2g and 0.4g, such stand-off distances may be further reduced to about 25 m and 14 m, respectively for 2000 kg charge weight. Observation in the near field, however, deserves further attention.

This information may be important from a strategic viewpoint. Important structures such as heritage buildings, strategically important and government buildings *etc.* may be protected by constructing a special barricade around the structure, maintaining such distance away from the same. The event of a blast outside the barricade may cause the demolition of the barricade, but the structure may survive as the effect of the blast is observed to die down very rapidly with distance.

5. Influence of shape or decay parameter 'b' does not seem to bear any recognizable influence on the overall response for the practical purpose as the variation of the same, in the practical range of interest, leads to an insignificantly small variation in impulse.

6. The limited results indicate that a solitary building may be used for further study and research under blast loading as it is subjected to more potential danger as compared to its similar counterpart in urban congested environment. However, the potential danger in an urban congested environment may primarily arise due to the possibility of an explosion with a very short stand-off distance and a simultaneous effect on many structures near the point of explosion.

7. The present investigation, therefore, prepares the background to assess the response of low-rise buildings due to blast. In the design codes of many countries, an adequate combination of blast loading with other loading is not included, as is done for seismic loading with dead and live loads. Thus, consideration of such loading is optional. The equivalency of blast and seismic loading established in terms of elastic as well as an inelastic response helps to incorporate and to assess the safety under blast loading in terms of considering equivalent seismic loading in the code-prescribed load combinations. Further, the feasible load-combinations involving dead load, live load and blast may be framed assessing the lateral force potential of a particular blast loading through conversion of the same to the equivalent seismic loading. In such to-be-framed design strategy, the extent of likely charge weight and stand-off distance may be regulated through the importance factor reflecting the impact of the damage or destruction relevant to class of systems. Such factors suggested in the currently practiced earthquake codes may be, with due re-appraisal in the context of explosive shock, used as a convenient guideline to achieve such end. The results embodied herein are urged to be re-visited addressing the time-dependent effects of blast on different parts of the structures such as roofs, sides and rear.

10. PROPOSALS FOR FUTURE RESEARCH

Observations of the present study may be reviewed through a more appropriate model of air-burst accounting for the impact of the same on the roof and on the walls of the structures (including side and rear) [42]. Response quantities need be evaluated assuming the structure to lie in a 'regular reflection' regime or a combination of 'mach' and 'regular' reflection depending upon the variation of height of burst and angle of reflection. Such studies should also incorporate the effect of strain rate as the strength of structural steel is known to increase at high strain-rate. Relevant curves for 'dynamic increase factor' with strain rate presented elsewhere [60] may be used for the same. Further, it intuitively appears that the damage due to burst action has a propensity to be closer to that due to earthquake for reinforced concrete (*R/C*) systems that experience stiffness and strength degradation under repetitive loading. Hence, conducting a similar investigation for *R/C* structures is deemed promising. Hysteresis model developed elsewhere [56] may appear useful to such end.

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