Probabilistic assessment of blast-induced progressive collapse in a seismic retrofitted RC structure

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ABSTRACT :

Extreme loading conditions such as man-made malicious actions, fires or natural events could induce local failure mechanisms (e.g., a loss of a member) which may trigger progressive collapse. By progressive collapse, it is intended disproportionate magnification of a minor failure event within the structure. The design or the assessment of a critical infrastructure needs to address the possibility of such an extreme circumstance taking place during its effective life-time. It is observed that blast-induced progressive collapse mechanisms involve non-linear structural behavior similar to that due to earthquakes.
This work focuses on probabilistic analysis of progressive collapse of a typical RC structure, induced by a blast event. The objective is to verify the effectiveness of seismic retrofitting schemes against explosions and the eventual progressive collapse. The probabilistic analysis is performed by taking into account the uncertainties in loading such as planar configuration and amplitude of the blast loading. A standard Monte Carlo simulation method is employed to generate various realizations of the uncertain parameters within the problem. For a given realization, various component-level dynamic analyses are preformed within a certain range of distance, in order to quantify and locate the damage induced by shock wave on structural elements. In the next step, a limit state analysis is performed, in order to investigate whether a progressive collapse mechanism forms under the acting loads.
The probabilistic analysis described above is being performed for the structure before and after it is subjected to a seismic retrofit scheme in order to highlight the efficiency of seismic retrofitting techniques against blast-induced progressive collapse. As the case study, a 5-storey reinforced concrete frame structure designed for gravity loading only is considered. As a possible retrofitting scheme, steel bracing installation is chosen. The braces in the retrofitted structure are designed to mitigate seismic risk. The mean annual rate of collapse considering both blast and earthquake for the structure before and after retrofit are compared.

KEYWORDS: multi-hazard assessment, progressive collapse, blast, Monte Carlo simulation, plastic analysis.

1. INTRODUCTION

In order to conduct design or assessment of strategic infrastructures, all possible critical events that could occur during their life-time need to be considered. This calls for the evaluation of the probability of collapse for the investigated structure via a multi-hazard procedure, in which all events that could produce significant damage are accounted for. In particular, it can be written:

$$P(C) = \sum_{A} P(C \mid A)P(A) \tag{1.1}$$

where A stands for a critical event, such as, earthquake, blast, etc., that is:

$$A \equiv EQ + Wind + Gas\ Explosion + Blast + MISC \tag{1.2}$$

Equation 1.1 is written, according to total probability theorem assuming that the critical events A are mutually exclusive and collectively exhaustive. Obviously, in Equation 1.1 some of the terms can be neglected if the rate of occurrence of the corresponding events is practically negligible. Hence, the multi-hazard acceptance criteria can be written as following:

$$P(C) = \sum_{A} P(C \mid A)P(A) \leq \nu_{dm} \tag{1.3}$$

where ν_{dm} represents the *de minimis* risk, which defines that risk below which society normally does not impose any regulatory guidance, which is in the order of 10^{-7}/year (Patè-Cornell, 1994).

2. A BI-HAZARD APPROACH CONSIDERING BLAST AND EARTHQUAKE

Herein, a particular case is studied in which the predominant critical events are blast and earthquake. In fact, these two hazards can coexist in the case of particularly strategic infrastructures located in a seismic zone. In this case, Equation 1.3 reduces to:

$$\nu_C = P(C \mid EQ)\nu_{EQ} + P(C \mid Blast)\nu_{Blast} \leq \nu_{dm} \tag{1.4}$$

where ν_C represents the annual rate of collapse and ν_{EQ} and ν_{Blast} stand for the annual rates of occurrence of earthquake and blast events of significance, respectively. $P(C|EQ)$ and $P(C|Blast)$ represent seismic and blast fragilities. The annual rate of an earthquake event of interest can be calculated using probabilistic seismic hazard analysis (PSHA) for the site of interest, where the considered structure is located. In this work, the blast and seismic fragilities are defined rather differently; that is, the seismic fragility is defined as the probability of collapse for a level of seismic intensity whereas the blast fragility is calculated as the probability of collapse given that a significant blast event takes place.

Both blast and earthquake loading conditions involve the activation of energy dissipation mechanisms and, as a consequence, both can be resisted employing ductility enhancing techniques, such as column wrapping or jacketing and steel bracing (NIST/GSA, 2001). Hence, it is interesting to investigate the extent to which seismic structural retrofit choices are able to improve structural performances under blast.

3. THE PROGRESSIVE COLLAPSE MECHANISMS

The most critical consequence of a blast event is normally a global progressive instability induced by local severe damage; this is usually referred to as the progressive collapse. Progressive collapse of structures is currently focusing the attention of many researchers in structural engineering community; in fact, especially in critical infrastructures, where exceptional events must be considered as possibly occurring during structure life-time,

progressive collapse need to be investigated and prevented. Hence, a number of research activities are conducted in order to study progressive collapse mechanisms (Bao et al. 2008, Yi et al. 2008).
In this study, the event of progressive collapse in the structure, given a blast event, is characterized by a Bernoulli variable that assumes the value of one in the case of progressive collapse and zero otherwise. Therefore, the probability of blast-induced progressive collapse can be calculated as the expected value of such Bernoulli variable over all possible blast scenarios. For instance, a blast scenario can be identified by the location in which the blast event can possibly take place and by the intensity of the blast charge.
The blast fragility, denoted by $P(C|Blast)$, is herein defined as the conditional probability for the event of progressive collapse given that a blast event takes place near or inside the strategic structure in question.
Let real vector $\underline{\theta}$ represent all the uncertain quantities of interest, related to structural modeling and loading conditions. Let the positive real number $p(\underline{\theta})$ represent the probability density function (PDF) for the vector $\underline{\theta}$. The $P(C|Blast)$ can be written as follows:

$$P(C \mid Blast) = \int I_{C|Blast}(\underline{\theta}) p(\underline{\theta}) d\underline{\theta} \tag{3.1}$$

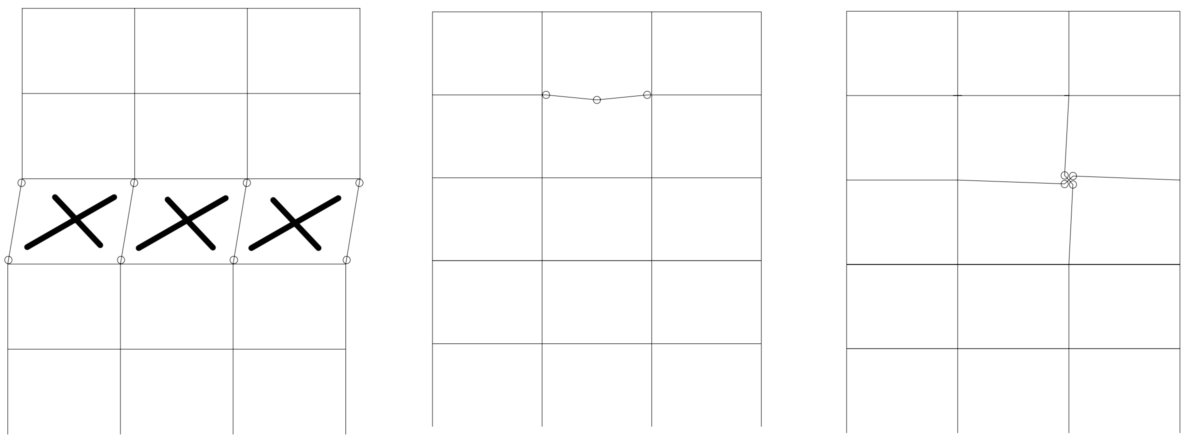
where $I_{C|Blast}(\underline{\theta})$ is an index function which is equal to unity in the case where $\underline{\theta}$ leads to blast-induced progressive collapse and zero otherwise. The probability of progressive collapse $P(C|Blast)$ can be then calculated using standard Monte Carlo (MC) simulation for generating N_{sim} samples $\underline{\theta}_t$ from PDF $p(\underline{\theta})$. Herein the vector $\underline{\theta}$ consists of the position of the blast charge with respect to the structure and the quantity of explosive. For each realization of the blast, the induced local damage is calculated based on component analysis (since the load rate is so rapid that each component reacts as if it was a single member). Based on the results of the component analysis, the set of members that have failed (or have experienced serious damage) can be identified and removed. A kinematic plastic limit analysis is then performed on the structure that has lost some of its elements to investigate whether it can still withhold the acting loads. The event of progressive collapse is identified by the ratio index $\lambda_C(\underline{\theta}_t)$ which is the factor by which the gravity loads should be multiplied in order to create a global collapse mechanism. The probability of blast-induced progressive collapse could be roughly estimated by the ratio of number of collapse cases to the total number of the considered configurations.

4. KINEMATIC PLASTIC ANALYSIS ON DAMAGED STRUCTURE

After identifying and removing the damaged elements, it should be verified whether the damaged structure can withstand the applied vertical loads. The plastic limit analysis can be performed in order to find the smallest *kinematically admissible* load for which the following conditions are satisfied.

1. Equilibrium conditions are satisfied.
2. Assuming that the non-linear behavior in the structure is concentrated at the element ends and the member ends are capable of developing their plastic moment, a sufficient number of plastic hinges are formed in the structure in order to transform the whole structure or a part of it into a collapse mechanism.

A kinematically admissible load corresponds to a mechanism in which both the external work e done by the applied forces on virtual deformations and the internal work u done by the ultimate moments M_P on the virtual rotations θ are positive. It should be noted that in the presence of the steel braces, the internal work u is incremented by amount of work done by the tensile yield force in the braces on the corresponding virtual elongation of the brace Δ. The mechanism corresponding to the smallest kinematically admissible load is calculated as the linear combination of a set of possible principal mechanisms in the structure, such a those outlined in Figure 1. The presence of the steel braces only affects the internal work done by the storey mechanisms. The internal work done by the braces in compression is herein neglected.



(a) storey mechanism (b) beam mechanism (c) joint mechanism

Figure 1 Principal mechanisms, as defined in (Grierson and Gladwell, 1971)

5. TOTAL RISK EVALUATION

Once the blast fragility is evaluated, it is implemented in Equation 1.4 in order to calculate the annual risk of collapse. The contribution of the seismic risk to Equation 1.4 is calculated by integrating the seismic fragility curve for the structure and the spectral acceleration hazard evaluated for the site of the project at a period close to the fundamental period of the structure. The seismic fragility can be calculated by implementing non-linear dynamic analysis procedure. The annual risk of collapse can then be compared with the *de minimis* threshold.
It should be noted that the annual rate ν_{blast} that a significant blast event takes place cannot be easily quantified, as it depends more on the socio-political circumstances and the strategic importance of the investigated structure. In the case of a non-strategic structure ν_{blast} can be in the order of 10^{-7}, causing the blast contribution to the annual risk of collapse to be negligible. In case of a strategic structure, ν_{blast} can be quite larger than 10^{-7}.

6. NUMERICAL EXAMPLE

As a numerical example, the methodology described in the previous section can be applied in order to calculate the annual risk for progressive collapse for a generic RC frame building, designed for gravity loading only and retrofitted with steel braces in its weak direction. The characteristics of the case-study structure are outlined in the following.

6.1. Structural model description

The case-study building is a generic five-story RC frame structure. The structural model is illustrated in Figure 2, presenting a plan of the generic storey.

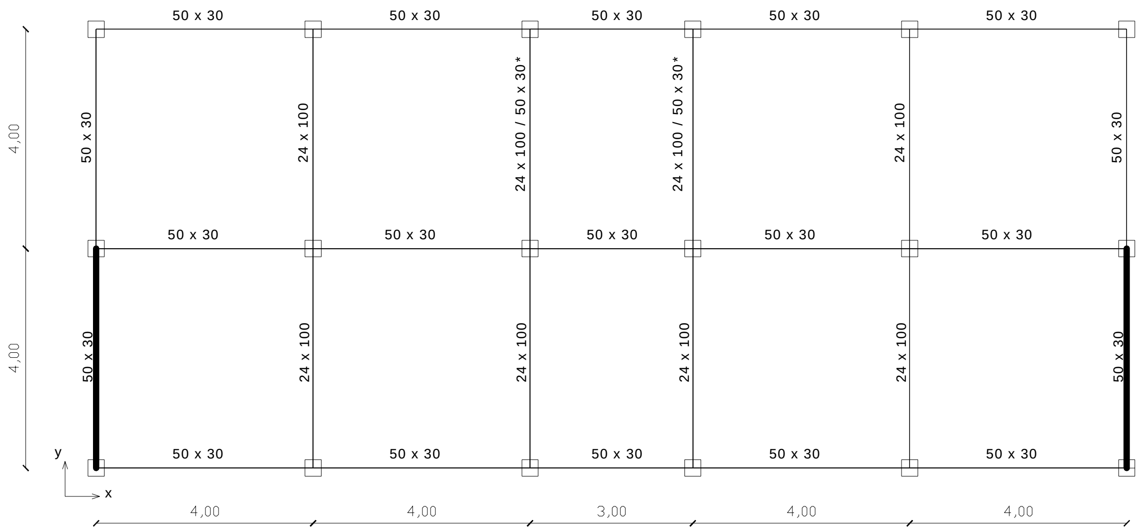


Figure 2 Storey view (dimensions in m)
Beam frame labels indicate the section dimensions in cm; column sections are all 30x30
**this frame represents both storey beams (24x100) and stair knee beams (50x30)*

Each storey is 3.00m high, except the second one, which is 4.00m high. The non-linear behavior in the sections is modeled based on the concentrated plasticity concept. It is assumed that the plastic moment in the hinge sections is equal to the ultimate moment capacity in the sections which is calculated using the Mander (Mander et al.. 1988) model for concrete and elastic-plastic model for steel rebar. The retrofit intervention consists of steel brace couples, installed in the panels indicated with a bold line in Figure 2, at every floor; in particular, from first to third floor braces with $10 cm^2$ of area and in the last two floors braces with $6 cm^2$ of area are considered.

6.2. Characterization of the uncertainties

As mentioned in the methodology, the uncertain quantities of interest in this study are the amount of explosive W and its position with respect to a fixed point within the structure denoted by R. Formally speaking, the vector of uncertain parameters contains two uncertain quantities: $\underline{\theta} = \{W, R\}$. The distance R is assumed to vary so that the explosion can take place both inside and outside the structure. Inside the structure it can take place on all floors. Outside the structure, it is assumed that the blast could take place on the second floor (the ground level) outside a standoff distance (assumed to be equal to 10 meters) from the structural perimeter.

For each simulation realization, first the center of explosion is simulated, assuming that the explosion takes place with a probability of 30% and 70% within and outside the structure, respectively. Once explosion realization occurs inside the structure, it is assumed that it is equally probable to take place on each of the 5 floors of the building. Then, after having determined the location of blast, the amount of explosive is simulated assuming that it can vary between 15 and 35 kg of equivalent TNT (simulating a backpack bomb) if the explosion takes place within the structure from the second to the fifth floor. If the explosion occurs on the first floor (the basement), the amount of explosive is assumed to vary between 200 and 500 kg of equivalent TNT (simulating a car bomb). Finally, when the explosion occurs outside the structure perimeter, the amount of TNT is assumed to vary between 15000 and 25000 kg of equivalent TNT (simulating a truck bomb) with 10% probability and between 200 and 500 kg of equivalent TNT with 90% probability. All uncertain quantities are assumed to be uniformly distributed.

6.3. Characterization of the parameters defining the local dynamic analysis

In order to identify the columns severely damaged by blast, a local dynamic analysis on the set of possibly afflicted columns has been performed. When the explosion occurs inside the structure, it is assumed that only the columns located on the same floor as that of the explosion are affected. This assumption is supported by the fact that the columns on the other floors are sheltered from the blast wave by the floor slab system. However, in the case of outside explosion, only the external columns directly exposed to the charge are assumed to be affected by the explosion (since the internal ones are sheltered by the perimeter walls).

For each of the columns hit by the explosion at the distance r from center the center of the charge, given the amount of explosive W, the reduced distance $Z=r/W^{1/3}$ is then calculated. Once Z is calculated, the triangular impact load parameters p_0 and t_{plus} can be obtained based on the semi-empirical formulas available in the literature (Henrych, 1979). It is further assumed that the intensity of the impact loading is uniform across the column length. Hence, the maxima for bending moment and shear force acting on the column are evaluated and compared with corresponding ultimate bending moment and shear values in order to determine whether the analyzed column fails. It should be noted that the applied load on the column is divided into two components, taking into account the angle in which the charge hits the column.

6.4. Blast fragility

A standard MC simulation is used to generate 500 blast scenario realizations, assuming that the structure is subject to its self-weight and to 30% of the characteristic live loads (equal to 2.0 kN/m^2). For each of these realizations, the collapse load factor λ_C is calculated and compared against the interval $1<\lambda_C<2$ typically leading to progressive collapse in the structure. The cumulative distribution function for the load factor denoted by $P(C|Blast)$ is plotted against λ_C in Figure 3, for the structure before and after the retrofit intervention. It can be observed from Figure 3 that, for the original structure there is around 18% probability that a blast event leads to progressive collapse, whereas for the retrofitted structure this probability reduces to 9%. The samples that lead to progressive collapse can provide further information about the blast vulnerable locations and the quantities of explosive that can be fatal. This kind of information is very helpful for blast-resistant design of strategic infrastructure.

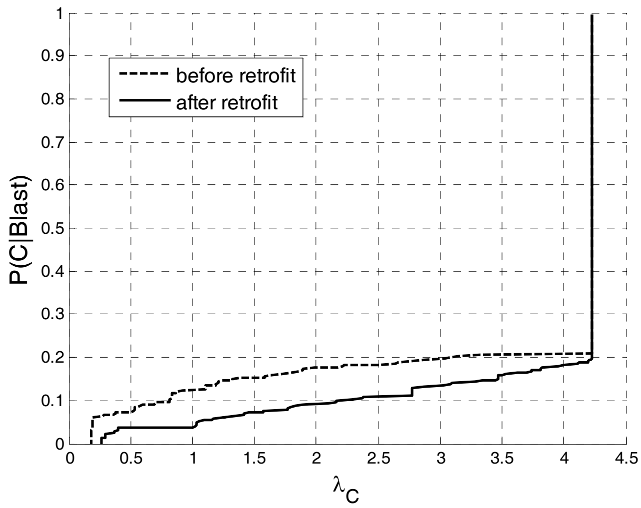


Figure 3 Blast Fragility

It should be noted that, both structures in the undamaged configuration (no column has failed) have the same value of $\lambda_C=4.22$. This is due to the fact that the collapse mechanism for the intact structure is actually a beam mechanism occurring on the last floor and does not involve any brace.

6.5. Evaluation of Seismic Risk

The seismic fragility for the case-study structure is calculated in two steps. In the first step, a non-linear static analysis is performed on the structure, modeled using the SAP (version 10) software based on the concentrated flexural plasticity concept, in order to obtain the static pushover curve for the control displacement at the top versus base-shear. The collapse threshold is marked at the point at which the first element goes in crisis (the severe damage limit state) .The equivalent elastic-perfectly plastic SDOF system, characterized by yield displacement, ultimate displacement and yield force, corresponding to the pushover curve for the entire structure is then approximated using a procedure recommended by the European seismic guideline. In the second step, a non-linear dynamic analysis procedure known as the multiple-stripe analysis (Jalayer 2003) is used in order to obtain the seismic fragility. A suite of 50 ground motion records are scaled to increasing levels of spectral acceleration and applied to the equivalent elastic-plastic SDOF system. The displacement response of the equivalent SDOF structure is then obtained using the multiple-stripe method. At each spectral acceleration level, the probability of failure (i.e., fragility) is estimated with the fraction of records that induce maximum displacement response larger than the ultimate displacement. The resulting fragility curves plotted in Figure 4, illustrate the conditional probability of failure as a function of the structural acceleration for the structure before and after retrofit.

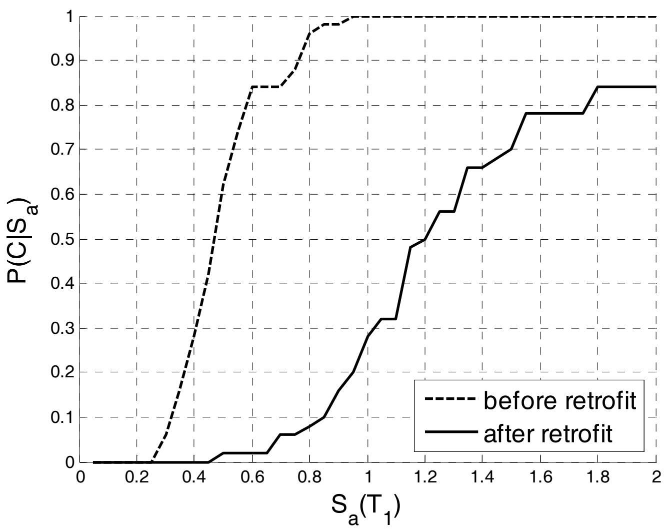


Figure 4 Seismic Fragility

In the next step, the seismic fragility is integrated with the hazard for spectral acceleration at a period close to the fundamental period of the structure (T=0.70 for the original structure and T=0.62 for the retrofitted structure) in order to calculate the annual rate of collapse due to earthquakes (the seismic risk). The annual rate of exceeding spectral acceleration has been extracted from INGV seismic maps assuming that the structure in question is located at Naples, Italy. The seismic risk for the original structure is equal to $5.8x10^{-4}$. For the retrofitted structure the seismic risk reduces to $3.8x10^{-5}$.

6.6. The annual rate of collapse

In order to calculate the annual rate of collapse, contributions from both blast and earthquake according to Equation 1.4 should be considered. However, as mentioned before, the annual rate ν_{blast} that a significant blast event takes place cannot be easily quantified. In order to get an idea about the effect of the retrofit intervention on the total risk, it is supposed that ν_{blast} is equal to $5x10^{-3}$ (roughly 60% probability that a significant blast event takes places in 200 years). Therefore, using the blast fragility and seismic fragility calculated in the previous sections, the annual rate of collapse can be calculated from Equation 1.4 as follows:

$$\nu_C = 5.8\times10^{-4} + 9.0\times10^{-4} = 1.5\times10^{-3} \quad \text{for the original structure} \qquad (6.1)$$
$$\nu_C = 3.8\times10^{-5} + 4.5\times10^{-4} = 4.9\times10^{-4} \quad \text{for the retrofitted structure}$$

It can be noted from the above expressions that the retrofit intervention has decreased the seismic risk, given by $P(C|EQ)\,\nu_{EQ}$, by more than an order of magnitude (reduction by a factor of 15). The retrofit intervention has also caused a reduction by a factor of 2 in the blast risk, given by $P(C|Blast)\ \nu_{Blast}$. The overall effect of the steel bracing installation is reduction in the annual rate of collapse by a factor of 3. It should be noted that in this case the bracings were designed according to seismic criteria only. In order to yield more significant reduction in the total risk, the bracings system should be optimized also with respect to the blast criteria. Nevertheless, the results verify the effectiveness of the steel bracings in mitigating the blast risks.

6. CONCLUSIONS

Objective of the present work is to present a methodology for calculating the annual risk of collapse for a civil infrastructure structure subjected to both seismic and blast threats, using a bi-hazard approach. Given that a blast event of interest takes place, the probability of progressive collapse is calculated using a MC simulation procedure, producing set of possible realizations of the blast scenario in terms of the charge location and intensity. The simulation procedure implements an efficient plastic limit state analysis, formulated and solved as a linear programming problem, to verify whether progressive collapse mechanisms are activated under the service vertical loads or not. This leads to a significant increase in computational efficiency compared to the use of classical finite element analysis. The probability of collapse given an earthquake event of interest can be calculated by integrating the seismic fragility of the structure and the seismic hazard for the site. Once the contributions of seismic and blast threats are evaluated, they can be summed up to yield the annual risk of collapse.

It has been shown that the retrofit intervention not only manages to reduce the seismic risk by a factor of around 15 but also it leads to a reduction in the blast risk by a factor of 2. This further confirms the hypothesis that seismic retrofit schemes can also be effective in mitigating the risk of progressive collapse due to an explosion. This result also emphasizes the importance of considering blast criteria in the rehabilitation design of strategic structures in seismic zones. In fact, the presented methodology can prove particularly useful as a design and/or retrofit tool. That is, the methodology is able to trace the blast scenarios that lead to progressive collapse and to identify the risk-prone areas within the infrastructure. Furthermore, this type of information can also be implemented in blast prevention strategies (e.g., limit or deny access to risk-prone zones).

The methodology presented herein evaluates a given retrofit strategy in terms of the annual rate of collapse. However, it can be extended to retrofit decision making in which different viable strategies need to be considered and compared. In other words, the presented approach could be implemented in an optimization problem that searches the most suitable retrofit intervention subjected to a set of constraints reflecting the cost and feasibility of each retrofit solution.

REFERENCES

Grierson, D. E. and Gladwell, G. M. L., (1971) "Collapse Load Analysis using Linear Programming", Journal of Structural Division, Proceeding of the American Society of Civil Engineers, Vol 97, No ST5, May, pp. 1561-1573

Bao, Y., Kunnath, S. K., El-Tawil, S. and Lew, H. S. (2008) Macromodel-Based Simulation of Progressive Collapse: RC Frame Structures, Journal of Structural Engineering, Vol. 134, No. 7, pp. 1079-1091

NIST/GSA (2001) Workshop on Application of Seismic Rehabilitation Technologies to Mitigate Blast-Induced Progressive Collapse September 10, 2001 Oakland, CA

Pate-Cornell, (1994) E., "Quantitative safety goals for risk management of industrial facilities", Structural Safety Vol 13 No 3, pp. 145-157

Mander, J.B., Priestley, J.N. and Park, R. (1988) Theoretical Stress-Strain Model for Confined Concrete, Journal of Structural Engineering, Vol 114, No. 8, pp. 1804-1826

Henrych, J, (1979) The Dynamics of Explosion and its Use, Elsevier, pp. 558

Jalayer, F., (2003) "Direct probabilistic seismic anysis: implementing non-linear dynamic assessments" PhD Thesis, Department of Civil Environment Engineering, Stanford University, pp. 150

Yi, W. J., He, Q. F., Xiao, Y. and Kunnath, S. K. (2008) Experimental Study on Progressive Collapse-Resistant Behavior of Reinforced Concrete Frame Structures, ACI Structural Journal, Vol. 105, No. 4, pp. 433-439