Assessment of Typhoon Fragility for Industrial Buildings Using a Probabilistic Model

ABSTRACT

In this paper, a fragility analysis methodology is developed for assessing the response of steel-framed industrial buildings exposed to extreme typhoon winds. The fragility is a conditional limit state probability, presented as a function of the 3-second gust wind speed, based on a relation between statistics of wind loads and building resistances. Six different baseline industrial buildings considering different roof shapes, geographic locations were investigated using a probabilistic approach. The fragility methodology described in this paper can be used to develop performance-based design guidelines for industrial buildings in high wind regions as well as to provide information on which to base structural safety or expected loss assessments.

INTRODUCTION

Typhoons have caused billions of dollars losses in recent years in Korea (Fry 2003), and low-rise industrial buildings are one of the most vulnerable structures to high wind hazards.

However, a handful of studies exist in the public domain to predict wind damage of the industrial buildings. Most published studies use regression techniques with post-disaster investigations or claim data to develop fragility curves (Cope 2004, Khanuri 2003). This approach cannot be used to develop fragility curves for the industrial buildings in Korea because very limited post-disaster and claim data are existed.

In this paper, a probabilistic model is presented to predict the wind-induced building damage. This model uses a Monte Carlo simulation engine that generates damage information for typical industrial buildings, using a component approach. The simulation compares probabilistic wind loads and the probabilistic capacity of vulnerable building components based on pressure chamber tests, to determine the probability of damage, and finally probabilistic damage of a whole building is identified over a range of assigned wind speed.

BACK GROUND OF STRUCTURAL FRAGILITY ANALYSIS

The probability of any limit state of a structure can be expressed in convolution integral form if the hazard is a continuous function of demand y (Li 2005):

$$P(LS) = \int_0^\infty Fr(y)gx(y)dy \tag{1}$$

where *Fr(x)* = fragility function of demand *y* expressed in the form of a cummulative distribution function (CDF) and *gx(y)* is hazard function expressed in the form of a probability density function.

The structural system fragility often has been modeled by a lognomal cumulative distribution function. The lognormal fragility model is given by,

$$Fragility = Fr(y) = \Phi[\frac{\ln(y) - m_R}{\zeta_R}] \qquad (2)$$

where $\Phi(\cdot)$ is standard normal probability integral, m_R is median capacity which is dimensionally consistent with demand *y*, and ζ_R is standard deviation of *ln(R)*. Fragilities can be used to identify a level of demand that a component or system (building) will withstand certain probability.

FRAGILITY MODEL FOR INDUSTRIAL BUILDINGS

Baseline industrial buildings

Fragility assessments were performed for industrial buildings with various roof types and exposure conditions subjected wind loads. In this study, six baseline industrial buildings were considered, designated Prototype 1 ~ 6 buildings.

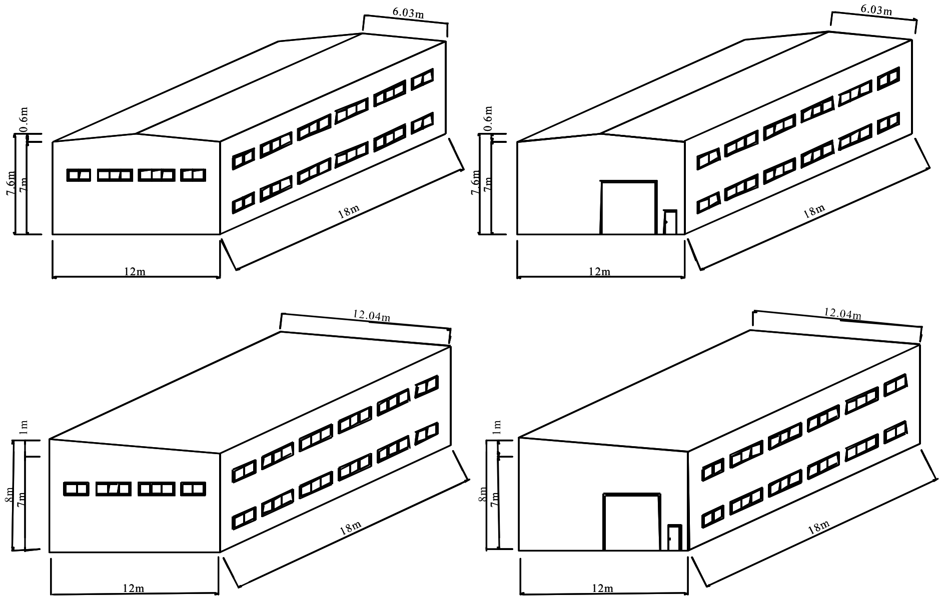

Fig. 1. Dimension and opening layouts for Prototype 1 and 4 buildings

Dimensions and detailed characteristics are shown in Table 1. The dimensions and opening layouts for the Prototype 1 and 4 buildings are shown in Fig. (1). Those for other baseline buildings are shown in reference (Lee and Ham 2007).

Table 1. Dimensions and characteristics of baseline industrial buildings

Properties	Prototype 1	Prototype 2	Prototype 3	Prototype 4	Prototype 5	Prototype 6
Plan Dimension	12m x 18m	12m x 18m	12m x 18m	12m x 18m	12m x 18m	12m x 18m
Roof Type	Gable	Gable	Multispan Gable	Monoslope	Gable	Gable
Roof Slope	6^o	10^o	10^o	5^o	6^o	10^o
Overhang	None	None	None	None	30cm	30cm

Resistance statistics of building components

Steel frame system incorporating sandwich panels is frequently used for industrial buildings in Korea. A series of pressure chamber tests was conducted to find out mean and COV values of the sandwich panels and windows. Fig. (2) shows pressure chamber tests conducted for sandwich panel systems. From the pressure chamber test, it was found that all the failures of the sandwich panel systems were caused by fastener pull-out or pull-over failure as shown in Fig. (3).

The statistics used for resistance capacity of typical sandwich panel systems and openings is shown in Table 2. For the statistics of entry and overhead roll-up doors, those values were obtained from the HAZUS-MH® model (FEMA 2006).

Fig. 2. Pressure chamber test (left; wall panels, right; roof panels)

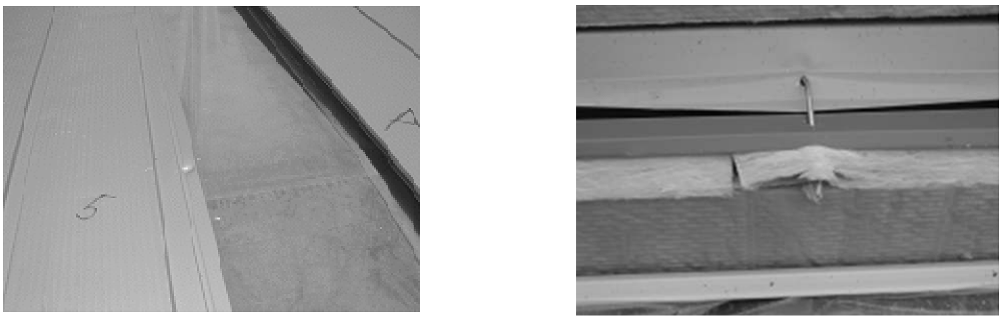

Fig. 3. Failure patterns of sandwich panels (left; wall panels, right; roof panels)

Table 2. Statistics of wind resistance capacities

Component	CDF	Distribution Parameter
Sandwich Roof Panel	Normal	Mean=3.25 kPa, COV=0.24
Sandwich Wall Panel	Normal	Mean=1.20 kPa, COV=0.21
Overhead Roll-Up Doors	Normal	Mean=1.20 kPa, COV=0.20
Entry Door	Normal	Mean=2.39 kPa, COV=0.20
Window	Normal	Mean=2.39 kPa, COV=0.20

Wind load statistics

In this study, the wind load acting on components and cladding for low-rise structures in ASCE 7-02 (ASCE 2003) is modified as follows:

$$W = \alpha \ q_h [GC_p - GC_{pi}] \tag{3}$$

where α = 0.8: factor to remove the safety factor embedded in ASCE 7-02 (Cope 2004), q_h = velocity pressure evaluated at mean roof height (h), GC_p = product of gust factor and external pressure coefficient, and GC_{pi} = product of gust factor and internal pressure coefficient. The velocity pressure evaluated at height (z) in ASCE 7-02 is given by:

$$q_z = 0.613 K_z K_{zt} K_d V^2 I \qquad \text{(unit: N/m}^2\text{)} \tag{4}$$

where K_z = the velocity pressure exposure factor, K_{zt} = the topographic factor, K_d = the wind directionality factor, V = the basic wind speed in m/s, and I = the importance factor.

Some of wind load statistics used in this study are shown in Table 3. For statistics of GC_p, information provided by references (Ellingwood and Tekie 1999, Lee and Rosowsky, 2005) is used to calculate mean-to-nominal and COV after calculating nominal external pressure coefficients for individual components by using ASCE 7-02.

Table 3. Summary of wind load statistics

Parameter	Category	Nominal	Mean	COV	CDF
K_z	Exposure B, 0.0m ~ 9.1m	0.70	0.71	0.19	Normal
	Exposure C, 0.0m ~ 4.6m	0.85	0.82	0.14	Normal
	Exposure C, 4.9m ~ 6.1m	0.90	0.84	0.14	Normal
	Exposure D, 0.0m ~ 4.6m	1.03	0.99	0.14	Normal
	Exposure D, 0.0m ~ 6.1m	1.08	1.04	0.14	Normal
K_d	Components & Cladding	0.85	0.89	0.16	Normal
GC_{pi}					
	Enclosed	0.18	0.15	0.33	Normal
	Partially Enclosed	0.55	0.46	0.33	Normal
K_{zt}			Deterministic (1.0)		
I			Deterministic (1.0)		

Monte Carlo simulation engine

A Monte Carlo Simulation engine was developed to simulate probabilistic wind loads and building component resistances. A flowchart of the developed model is shown in Fig. (4), in which shading identifies tasks within the nest loop. For each wind speed, this model simulates 3-second gust wind speed, velocity exposure factor, wind directionality factor, pressure coefficients, and component resistances by sampling from the assumed normal distributions, and this model compares wind load capacities and component resistances. These comparisons are repeated 10,000 times to develop the component fragility curves using predefined damage state definitions for each wind speed. In current Monte Carlo simulation engine, component resistances are assumed to be independent.

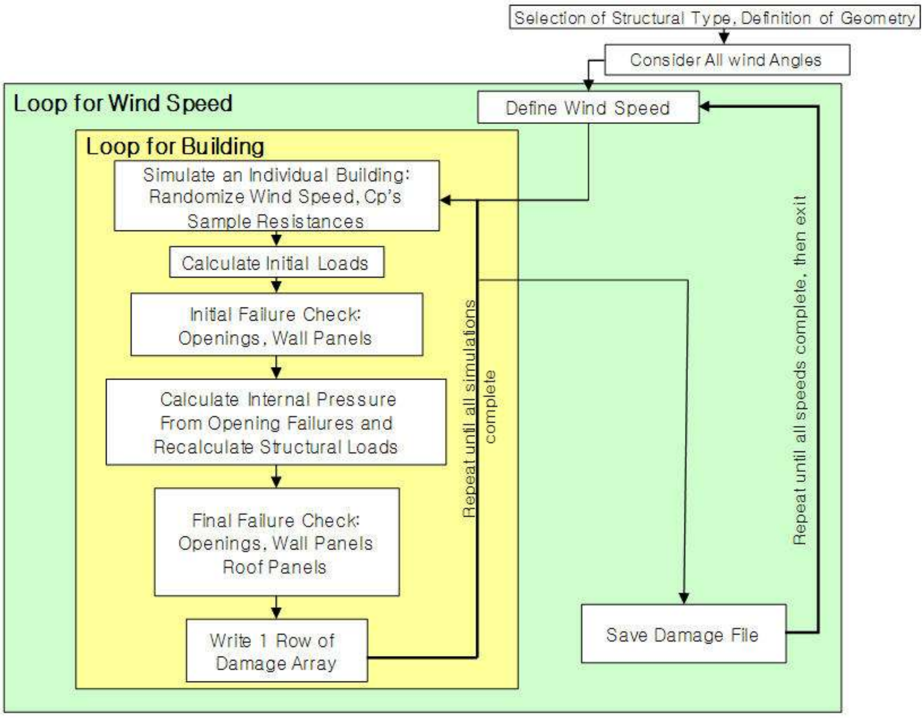

Fig. 4. Flow chart of Monte Carlo simulation engine

Validation of Monte Carlo simulation engine The Monte Carlo Simulation Engine (MSE) described above must be validated before developing typhoon fragilities for industrial buildings. The validation is performed by comparing fragility predictions to post disaster survey data following hurricane in the States since those data are not currently available in Korea.

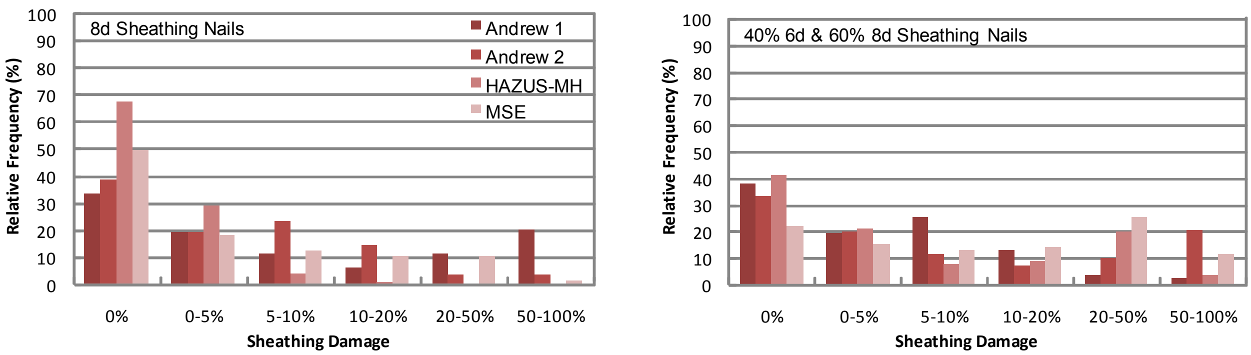

Fig. 5. Comparison of modeled and observed roof damage state to single story gable house

Fig. (5) illustrates the results of the aggregation for failure of roof sheathings on single story gable roof house and makes a comparison of the predicted damages with observed in post-disaster damage survey of Hurricane Andew (NAHB, 1993). In the figure, the MSE and HAZUS-MH® models used same fastener layouts (i.e., 100% 8d sheathing nails and 40% 6d & 60% 8d sheathing nails) for the simulations. It can be observed that the damage prediction obtained by developed current Monte Carlo simulation engine is reasonably match well with the post-disaster survey results compared with the damage prediction by HAZUS-MH® model.

TYPHOON FRAGILITIES

This paper presents selected results of a study on Prototype 1 ~ 6 buildings because of space limitations. The building damage states associated with the industrial buildings are defined in Table 4. Note that no frame failures are modeled, with the entire performance of the building governed by the performance of the cladding.

Table 4. Damage state for industrial building

Damage State	Damage Descriptions	Entry/Overhead Door Failure	Sandwich Wall Panel Pull-Over Failure	Sandwich Roof Panel Pull-Out Failure
0	No Damage	No	No	No
1	Minor Damage	One Door	One Panel	One Panel
2	Moderate Damage	>10	>10	>10
3	Severe Damage	>20	>20	>20
4	Destruction	>33	>33	>33

Lognormal fragility model

Fig. (6) presents fragility curves of Prototype 1 building for the exposure B and D conditions. The fragility can be seen most simply as the limit state exceedence probability for a given 3-second gust wind speed at 10m above the ground in exposure C. In the figure, symbols represent the calculated fragility curves while lines are used to represent the lognormal cumulative distributions obtained by best fit analysis. Fig. (6) shows that the lognormal cumulative distribution provides a good fit to the calculated structure fragility curves. Table 5 summarizes the best-fit lognormal parameters for the fragilities of the Prototype 1 building.

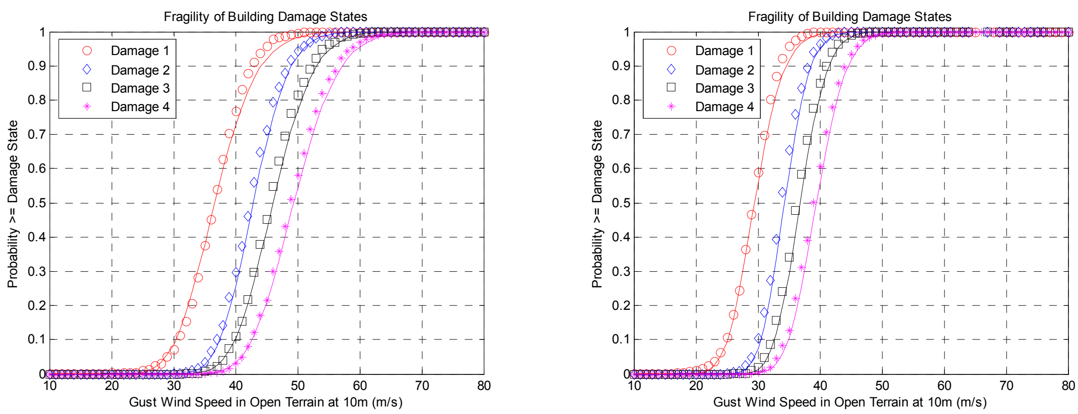

Fig. 6. Fragility curves for Prototype 1 building (left; exposure B, right exposure D)

Table 5. Lognormal parameters for Prototype 1 building

Baseline Structure	Exposure Condition	Damage State	Best-Fit Lognormal Parameters m_R	ζ_R
Prototype 1	Exposure B	Level 1	3.5581	0.1452
		Level 2	3.7219	0.1048
		Level 3	3.7943	0.1107
		Level 4	3.8644	0.1083
	Exposure D	Level 1	3.3823	0.1251
		Level 2	3.5348	0.0923
		Level 3	3.6057	0.0927
		Level 4	0.6747	0.0958

Component and building fragilities

Fig. (7) shows fragility curves for individual building components and a whole building. Prototype 2 building and exposure condition B are used to develop these fragility curves. As observed in Table 2, fragility curves for wall panels and openings show much higher values than those of roof panels.

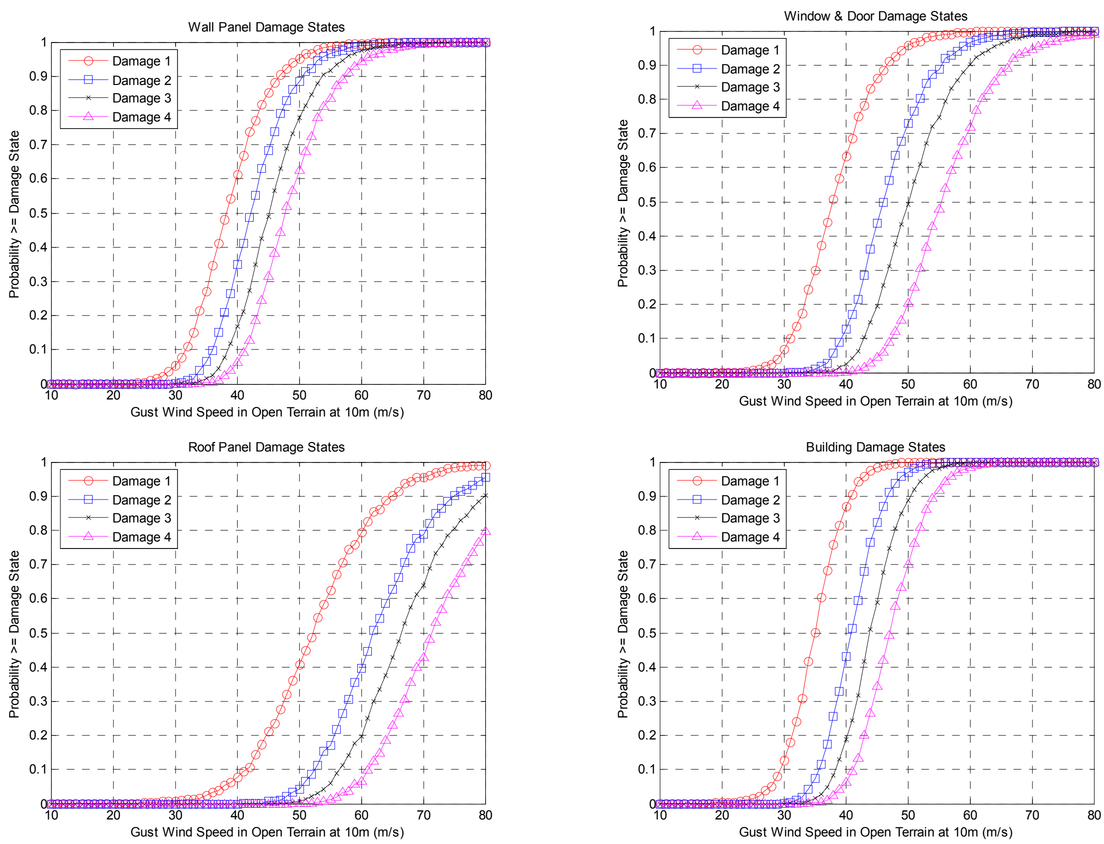

Fig. 7. Fragility curves for Prototype 2 building (exposure B condition)

Effect of building geometry

Fig. (8) presents a comparison of fragility curves for the six different baseline buildings. For this comparison, exposure B condition and damage state 4 are used. As shown in Fig. (8), fragilities of wall panels and openings show that there are negligible differences existed among six different prototype buildings since wall panel and opening layouts are similar to each other.

However, it can be clearly observed that fragilities for the roofs with overhangs (Prototype 5 and 6) are higher than those for roofs without overhangs (Prototype 1 and 2). Also, it can be found that the multi-gable roof (Prototype 3) system is more vulnerable to wind loads than the gable roof system (Prototype 2) for same roof angle (i.e., 10^{o}).

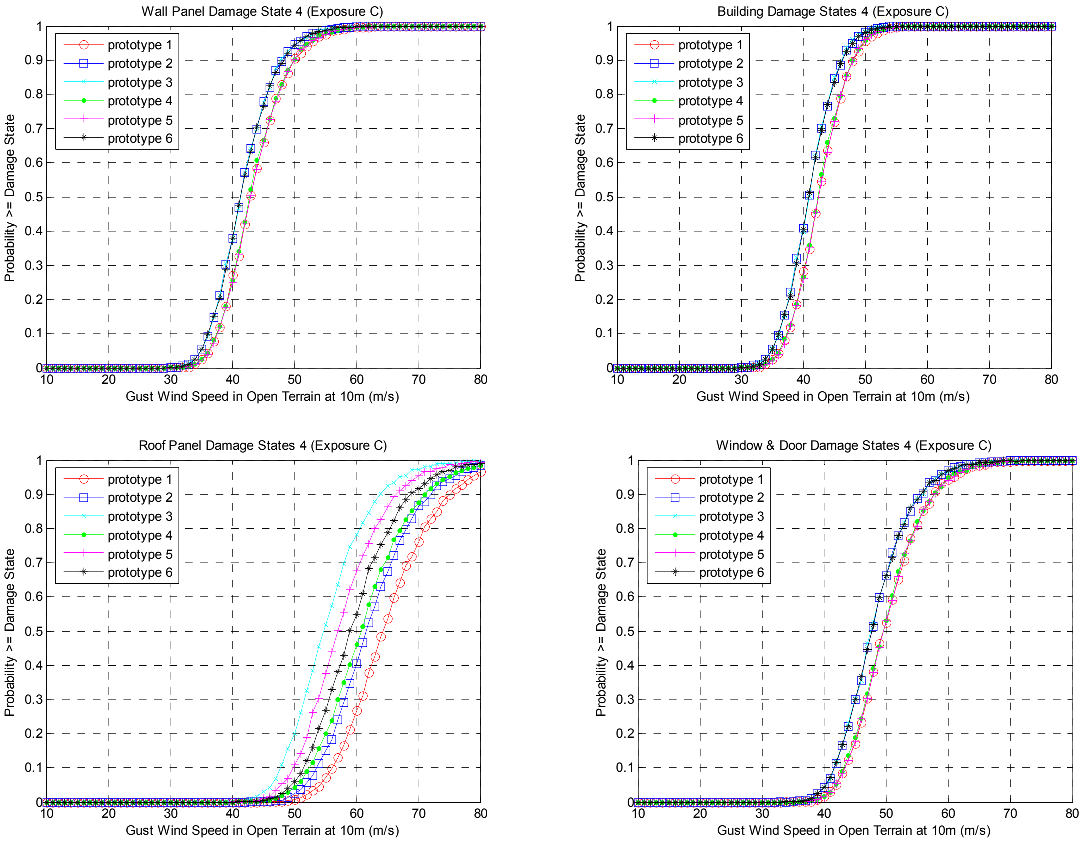

Fig. 8. Fragility curves for Prototype 1 ~ 6 buildings
(exposure condition B and damage state 4)

CONCLUSIONS

This paper presented selected results of a study to develop fragility curves for low-rise industrial buildings built in high wind region. A Monte Carlo simulation engine compares probabilistic wind loads and the probabilistic capacity of vulnerable building components based on pressure chamber tests, to determine the probability of damage. Use of this methodology could lead to more predictable building performance and facilitate the introduction of performance-base engineering for industrial building construction, improving the utilization of steel-frame and sandwich panels leading to more reliable and economical design of steel-framed industrial building construction. Also, the methodology can be used to develop a risk assessment tool, which can evaluate the potential impact of a natural hazard in public planning and mitigate the consequent economic losses and social disruption.

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