Fragility Assessment for Cladding of Industrial Buildings Subjected to Extreme Wind

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Abstract

This paper presents a fragility assessment for the cladding in steel-framed constructions built in extreme wind speed regions. A fragility assessment methodology is developed to assess the performance of sandwich wall/roof panels and openings subjected to wind-induced building pressures. The majority of industrial buildings in Korea are of steel-framed construction. A review of the performance of industrial buildings after recent typhoons Maemi, Kompasu and Bolaven has shown that most of wind damage originated in the building envelope. The goal of this study is to develop a fragility model for building cladding, using available resistance test data, developed wind load statistics, and a recent building code. Six baseline structures considering different roof shapes, building sizes and geographic locations were investigated using a fully probabilistic Monte Carlo simulation engine. The fragilities of each baseline building considering four different levels of damage were developed as a function of a 3-second gust wind speed. The fragility assessment methodology described in this paper can be used to measure the performances of industrial buildings in extreme wind speed regions, as well as to provide information for risk and loss assessments.

Keywords: extreme wind speed; fragility; industrial building; cladding; Monte Carlo simulation

1. Introduction

Steel-framed construction has been widely used for industrial buildings, such as factories, workshops, and warehouses. In Korea, much of this steel-framed construction utilizes folded beams and columns as its structural members, and sandwich or metal panels as its claddings.

In Korea, typhoons have caused billions of dollars (USD) in losses in recent years (Fry, 2003; Park, 2011), and have exposed the increasing vulnerability of low-rise industrial buildings that use sandwich panels, as shown in Fig.1.

Typhoon damage to an industrial building results in not only direct losses, such as the building itself, but also triggers indirect losses, such as business interruptions due to damages to manufacturing facilities or stored products (Boughton et al., 2011).

In structural design, protection of building occupants against injury or loss of life is of paramount importance. Thus, the main objective of current codes and standards is to prevent building failures during extreme events that lead to loss of life.

While this objective has been well achieved for building frames in Korea subjected to extreme winds, the indirect losses resulting from cladding failures have become unacceptable. These indirect losses include economic losses and social disruption. It has become apparent that buildings designed under the current codes, although satisfying the life safety objective, may not meet other expectations of building owners and occupants.

Performance-based engineering (PBE), based on the probability of any limit state (LS), is a new paradigm that is gaining worldwide momentum (Herbin, 2008). It is motivated by a desire to add value to the building construction process, by ensuring that the building meets the expectations of the building owners and occupants.

The performance of an industrial building is affected by many variables and sources of uncertainty. Therefore, it is crucial to identify key parameters that influence the fragility of an industrial building, as this
identification provides the basis for the improvement of building performance under extreme winds (Li, 2006).

However, only a handful of studies have been carried out to develop wind fragility curves for the claddings of industrial buildings (Garcia, 2008; Zhao and Gu, 2011). Most wind fragility curves have been developed empirically or analytically. These results cannot be used directly in Korea, since cladding resistances and construction methods are different from those of other countries. Also, the empirical approach cannot be applied to develop fragility curves for industrial buildings in Korea, because very limited post-disaster and claim data are available and observational data tend to be highly specific to their source situations, and are very sparse in the domain representing more extreme wind events.

In this paper, a probabilistic model is presented to predict wind-induced building damage. This model uses a Monte Carlo simulation engine that generates damage information for typical industrial buildings, using a fully probabilistic approach. The simulation compares probabilistic wind loads and the probabilistic capacities of vulnerable building components, which are obtained from pressure chamber tests and literatures. In this methodology, individual fragilities of an envelope system are estimated first, and then the probabilistic damage of a whole building is identified.

2. Background 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, 2006):

\[
P_f(\text{LS}) = \int_0^\infty Fr(y)gx(y)dy
\]  

(1)

where \( Fr(y) \) = fragility function of demand \( y \) expressed in the form of a cumulative distribution function (CDF), and \( gx(y) \) is hazard function expressed in the form of a probability density function (PDF).

Fragility function describes the probability of failure, conditioned on the load, over the full range of loads to which a system might be exposed (Jiang et al., 2012). Approaches to developing fragility curves can be classified into four broad categories: judgmental, empirical, analytical, and hybrid (Schultz et al., 2010). Judgmental approaches are based on expert opinion or engineering judgment. Empirical approaches are based on observations. Analytical approaches are based on models. Hybrid approaches combine two or more of the other approaches.

In this study, wind fragility curves are developed by using a Monte Carlo simulation-based analytical approach. This approach generates samples, or realizations, of the random demand and capacity variables from their specified distributions, and evaluates whether wind-induced failure occurs, by using the following limit state function, \( G \):

\[
G(R, W) = R - W
\]

(2)

where \( R \) = resistance of a building component, and \( W \) = wind load acting on the component. This process is repeated many thousands of times in a Monte Carlo simulation engine, and the probability of failure is approximated by the fraction of failures conditional upon wind load variables.

The shape of a fragility curve describes the uncertainty of the system's capacity to withstand a load, or alternatively, the uncertainty in a load to cause failure of the system.

Structural fragility has often been modeled by a log-normal cumulative distribution function, since most natural loads follow a log-normal form. The log-normal fragility model (Straub and Kureghian, 2008) is given by,

\[
\text{Fragility} = Fr(y) = \Phi[\ln(y) - m_R / \xi_R]
\]

(3)

where \( \Phi(\cdot) \) is standard normal probability integral, \( m_R \) is median capacity which is dimensionally consistent with demand \( y \), and \( \xi_R \) is the standard deviation of \( \ln(R) \).

3. Wind Fragility Model

3.1 Baseline Industrial Buildings

Fragility assessments were performed for industrial buildings with various roof types subjected to different wind load conditions.

In this study, six baseline industrial buildings were considered, which were designated as Prototypes 1 ~ 6. Dimensions and detailed characteristics are shown in Table 1. The dimensions and opening layouts for Prototype 1 are shown in Fig. 2. The characteristics of Baseline Buildings are shown in Table 1.

| Property | Prototype 1 | Prototype 2 | Prototype 3 | Prototype 4 | Prototype 5 | Prototype 6 |
|----------|-------------|-------------|-------------|-------------|-------------|-------------|
| Roof Type | Gable       | Gable       | Flat        | Multi Gable | Gable       | Flat        |
| Roof Slope | 6°          | 10°         | 5°          | 10°         | 6°          | 5°          |
| Eave Height | 7.0 m       |             |             |             |             |             |
| Overhang  | None        | 30cm        | None        |             |             |             |
| Plan      | 12 m x 18 m (for Prototype 1 ~ 5) | 12 m x 18 m (for Prototype 1 ~ 5) | 12 m x 18 m (for Prototype 1 ~ 5) | 12 m x 18 m (for Prototype 1 ~ 5) | 12 m x 18 m (for Prototype 1 ~ 5) | 12 m x 18 m (for Prototype 1 ~ 5) |
| Dimension | 18 m x 24 m (for Prototype 6) | 18 m x 24 m (for Prototype 6) | 18 m x 24 m (for Prototype 6) | 18 m x 24 m (for Prototype 6) | 18 m x 24 m (for Prototype 6) | 18 m x 24 m (for Prototype 6) |

Fig. 2. Opening Layouts for Prototype 1
Prototype 1 are depicted in Fig.2., and those for the other baseline structures are shown in the publication of Lee and Ham (2011).

3.2 Building Components and Resistance Statistics

In Korea, a steel-framed system incorporating glass-wool insulated sandwich panels is typically used for industrial buildings, since these panels are recognized and certified as being fire-proof. These sandwich panels have a unit width of 1,000 mm and are of various thicknesses. Roof panels are thicker than wall panels, since wind loads on roofs are higher than those on walls. The configurations of the wall and roof panels applied in this study are shown in Table 2.

| Sandwich Panel | Width  | Maximum Thickness | Steel Skin |
|----------------|--------|-------------------|------------|
| Wall           | 1,000 mm | 75 mm             | 0.5T       |
| Roof           | 1,000 mm | 100 mm            | 0.5T       |

A series of pressure chamber tests, following the dynamic wind uplift protocol of the National Research Council Canada (Baskaran et al., 2006), was conducted to obtain the resistances of the sandwich panels. Each panel system was tested repeatedly 8 times to estimate its mean resistance and coefficient of variation (COV).

Fig.3. Layout of Sandwich Wall Panels System (unit: mm)

Fig.4. Layout of Sandwich Roof Panel System (unit: mm)

Fig.5. Failure Patterns of Sandwich Panel Systems (Left: Wall, Right: Roof)

Figs.3. and 4. depict the layouts of the wall and roof panel systems, respectively, which were installed in a pressure chamber. The pressure chamber used in this study had dimensions of 7,530 mm x 4,200 mm. The panels were attached directly to C-Shape purlins or girths, using #6 self-drilling screws. The typical spacing of the screws was used in the layouts, following the average spacing described in the technical installation specifications of the sandwich panel industry (Kirin, 2005).

3.3 Wind Load Statistics

In this study, the description for the wind load acting on the components and cladding for low-rise structures in ASCE 7-05 (2006) is modified as follows:

\[ W = \alpha \cdot q_h [GC_p - GC_{pi}] \]  

(4)
where $\alpha$ = factor that removes the safety factor embedded in ASCE 7-05 (Cope, 2004), $q_s$ = 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 7-05 is given by:

$$ q_z = 0.613 K_s K_a K_d V^2 I $$

where $K_s$ = the velocity pressure exposure factor, $K_a$ = the topographic factor, $K_d$ = the wind directionality factor, $V$ = the basic wind speed in m/s, and $I$ = the importance factor.

Some of the wind load statistics used in this study are shown in Table 4. The statistics of $K_s$ and $K_d$ were obtained from ASCE 7-05 and a previous Delphi study of wind parameters (Ellingwood et al., 2004). The mean values of $K_s$ for elevations higher than 6.1 m in exposure categories C and D were obtained using the nominal-to-mean values obtained from lower elevations. For the statistics of $GC_p$, the information provided by Lindt and Rosowsky (2005) was used to calculate nominal-to-mean and COV values, after calculating the nominal external pressure coefficients for individual components by using ASCE 7-05. In this study, $K_s$ was made equal to unity, so as not to make the results dependent on the local topography surrounding the building. Additionally, the value of $I$ is also assigned as unity, considering that it is used to scale loads according to the importance of the structure, but this factor does not assist in the determination of actual loads during an extreme wind event. Note that each random parameter in Eq. (4) is modeled by a normal distribution; however, the product of the parameters follows a log-normal distribution (Datin and Prevatt, 2009), like actual wind loads.

Table 3. Summary of Wind Resistance Statistics

| Component                | Distribution | Statistic Parameters |
|--------------------------|--------------|----------------------|
| Sandwich Wall Panel      | Normal       | Mean = 1.20 kPa, COV = 0.21 |
| Sandwich Roof Panel      | Normal       | Mean = 3.25 kPa, COV = 0.24 |
| Overhead Roll-Up Doors   | Normal       | Mean = 1.20 kPa, COV = 0.20 |
| Dimension: 5.000 mm x 4.000 mm |             |                      |
| Entry Door               | Normal       | Mean = 2.39 kPa, COV = 0.20 |
| Dimension: 900 mm x 1,800 mm |             |                      |
| Window                   | Normal       | Mean = 2.39 kPa, COV = 0.20 |
| Dimension: 833 mm x 900 mm |             |                      |

Table 4. Summary of Wind Load Statistics

| Parameter | Category | Nominal | Mean | COV | CDF     |
|-----------|----------|---------|------|-----|---------|
| $K_s$     | Exposure B, 0.0 m ~ 9.1 m | 0.70    | 0.71 | 0.19| Normal |
|           | Exposure C, 4.9 m ~ 6.1 m | 0.90    | 0.84 | 0.14| Normal |
|           | Exposure C, 6.1 m ~ 9.1 m | 0.90 ~ 0.98 | 0.84 ~ 0.92 | 0.14| Normal |
|           | Exposure D, 4.9 m ~ 6.1 m | 1.08    | 1.04 | 0.14| Normal |
|           | Exposure D, 6.1 m ~ 9.1 m | 1.08 ~ 1.16 | 1.04 ~ 1.12 | 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 |
| $I$       | Deterministic (1.0) |         |      |     |         |

Fig. 6. shows the layout of the building claddings (i.e. wall/roof panels, windows, entry door and overhead roll-up door) of Prototype 1. Wind loads acting on individual cladding components were calculated for each tributary area of the components by using Eq. (4).

3.4 Monte Carlo Simulation Engine

A Monte Carlo simulation (MCS) engine was developed to simulate probabilistic wind loads and building component resistances and to calculate the limit state function given in Eq. (2). Fig. 7. shows the flowchart of the developed model. 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 then compares wind load capacities and component resistances. These comparisons are repeated 10,000 times to develop system fragility curves, under predefined damage state definitions for each wind speed.

In the current Monte Carlo simulation engine, component resistances are assumed to be independent. In this Monte Carlo simulation engine, the failure of a cladding component can be divided into direct and indirect failures, whose examples are shown in the fault trees of Fig. 8. In the fault trees, and symbols represent AND and OR gates, respectively.

Once the first panel fails by the difference of wind load capacity and cladding resistance, the internal pressure conditions change from those of an "enclosed" structure to those of a "partially enclosed" structure, and the failure probabilities are re-computed using the new pressure coefficients. The system failure probability (fragility) for each wall, roof and opening system at a given wind speed is then calculated using Eq. (6). This procedure is repeated for wind speeds ranging from 10 m/s to 80 m/s.
where \( F_{\text{system}}(N_f \geq j | V) = \sum_{i=j}^{N} F_{\text{System}}(N_f = i | V) \) \( (6) \)

4. Wind Fragility of Industrial Building

This paper presents selected results of a study of Prototypes 1 ~ 6. To develop a fragility curve, the damage state (DS) must be defined. In this study, damage states of industrial buildings were categorized into 4 distinct damage states defined in Table 5. Note that no frame failures are modeled, with the entire performance of the building governed by the performance of the cladding.
4.1 Validation of Log-Normal Fragility

Fig. 9. presents fragility curves for individual opening, wall and roof systems, and for the building system in exposure category D, as assigned in ASCE 7-05. The fragility curves of a building system refer to the damage of the entire building, while fragility curves of opening, wall and roof systems are related to specific parts of the building envelope. The fragility of an entire envelope (i.e. building system fragility) is approximated by assembling the individual system fragilities, by calculating the probability that at least one fragility of the individual system occurs.

Fragility can be seen most simply as the limit state exceedance probability for a given 3-second gust wind speed at 10 m above the ground in Exposure C. In Fig. 9., ◯, □, ◊ and ✽ symbols represent the calculated fragility curves, while lines are used to represent log-normal cumulative distributions obtained by best-fit analysis. It can be seen that the log-normal distribution demonstrates well the general behaviors of fragilities obtained by the Monte Carlo simulation engine. Table 6. summarizes the best-fit log-normal parameters of the building system fragilities for Prototype 1.

Table 6. Lognormal Parameters of Building System Fragilities for Prototype 1

| Baseline Structure | Exposure Category | Damage State | Best-Fit Lognormal Parameters |
|-------------------|-------------------|--------------|-----------------------------|
|                   |                  |              | $m_\theta$ | $\zeta_\theta$ |
| Prototype 1       | D                | 1            | 3.382     | 0.125          |
|                   |                  | 2            | 3.535     | 0.092          |
|                   |                  | 3            | 3.606     | 0.093          |
|                   |                  | 4            | 3.675     | 0.096          |

4.2 Characteristics of Wind Fragility of Low Rise Industrial Buildings

In Fig. 9., opening and wall systems are more vulnerable than the roof system. Also, the uncertainty in the resistance capacity of the roof system is larger than those of other individual envelop systems, because the fragility curves of the roof system take the form of broader S-shape functions.

Fig. 10. presents a comparison of fragility curves of building systems for two different exposure conditions (i.e. Exposures B and C). Prototype 2 and Damage State 1 are selected for this comparison. Effects of building exposure can be observed, because the mean value of the exposure factor, $K_z$, increases from 0.71 to 0.88 when the exposure changes from B to C, and the exceedance probability of failure in Exposure B is 76% when the wind speed is 40m/s, while in Exposure C the same building has a larger exceedance probability of failure (98%).

Figs. 11. and 12. show the effects of roof geometries. Since the layouts of wall and opening systems for Prototypes 1 ~ 5 are similar, only the roof system fragilities are shown in these comparisons.
Fig.11. compares the fragility curves of Prototypes 2 and 4 in the Exposure C condition and Damage State 3. In Fig.11. it can be seen that the multi-gable roof system is more vulnerable to wind loads than the gable roof system for the same roof angle (i.e. 10°), because the external pressure coefficients ($GC_p$) of the multi-gable roof are greater than those of the gable roof. Hence, when the wind speed is 50 m/s, the exceedance probability of failure of the multi-gable roof system is 38%, while that of the gable roof system is 10%.

Fig.12. shows the effects of roof overhang on roof system fragility for Prototypes 1 and 5 in the Exposure B condition and Damage State 4. The roof system with an overhang is more vulnerable than that without an overhang, because the external pressure coefficients ($GC_p$) are different for roof zones of a roof with an overhang. Thus, when the wind speed is 60 m/s, the exceedance probability of failure of the roof system is changed from 13% to 22%, as overhang is installed on the gable roof.

Fig.13. demonstrates the effects of building size on wind fragility, by comparing Prototypes 3 and 6, in the Exposure C condition and Damage State 1. In Fig.13., the bigger building (i.e. Prototype 6) is slightly more vulnerable than the smaller one (i.e. Prototype 3), although these two buildings have the same mono-slope roof. This increased vulnerability is attributed to two factors: increase in building height, and more building corner and edge areas. The mean value (0.88) of the exposure factor, $K_z$, of Prototype 6 is slightly larger than that (0.87) of Prototype 3, because of the slightly increased mean building height. Also, the extreme external pressures on the corner and edge areas of Prototype 6 are a little higher than on those of Prototype 3.

4.3 Implementation of Fragility
Developed fragility curves can be used to measure the performance of a building system subjected to extreme wind hazard. As an implementation of the developed wind fragility curves, the probability of failure, $P_f(\Delta S)$, under possible typhoon winds is determined for two representative city areas of Korea (i.e. Incheon and Busan cities), by convolving an average value of building system fragilities of Prototypes 1 ~ 6 and typhoon hazard functions, as shown in Eq. (1). To evaluate typhoon hazard function,
g(x,y), information by Lee and Ham (2011) was used.

In this analysis, the probability of failure of a 100 year return period is estimated with the fragility obtained in the Exposure D condition and Damage State 1.

The results of this analysis are shown in Table 7. The probability of failure of an industrial building in Pusan City is twice that of a building in Incheon City. This result suggests that if the safety level of failure probability is set to 10%, risk mitigation efforts should be focused on these kinds of industrial buildings in Pusan City.

5. Conclusions

This paper presented selected results to develop fragility curves for low-rise industrial buildings built in extreme wind speed regions. Six baseline industrial buildings were considered as representative of typical small-sized industrial constructions in Korea.

A Monte Carlo simulation engine was developed to estimate wind fragility curves, by comparing probabilistic wind loads and probabilistic capacities of vulnerable building components, which were obtained from pressure chamber tests and literatures.

The results validated the log-normal distribution model for determining the wind fragility of an industrial building. The results were also used to analyze the effects of exposure, roof geometry and building size on building and component fragilities. Fragility presented herein was convoluted with typhoon hazard functions to evaluate the failure probabilities of buildings in two different cities.

This fragility methodology can be used to predict building performance, and to facilitate the introduction of performance-based engineering to 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 to mitigate the consequent economic losses and social disruption.

In future research, the developed fragility curves will be validated when post-disaster survey data are fully available in Korea.

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Table 7. Failure Probability of an Industrial Building

| City       | Fragility Information | Wind Hazard Information | Probability of Failure |
|------------|-----------------------|-------------------------|------------------------|
|            | Exposure Category     | Damage State            | Return Period          | 3s Gust Wind Speed for 100 Year Return Period | V_{100} | P_{f} (LS) |
| Incheon    | D                     | 1                       | 100 year               | 34 m/s                             | 6.3 %   |
| Pusan      | D                     | 1                       | 100 year               | 44 m/s                             | 15.5 %  |

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