RISK ANALYSIS OF A POTENTIAL VERTICAL SHELTER: A DEMONSTRATIVE EXAMPLE

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RISK ANALYSIS OF A POTENTIAL VERTICAL SHELTER: A DEMONSTRATIVE EXAMPLE

Introduction

In a previous portion of this research (4), a systematic and comprehensive method of evaluating the safety of vertical shelters was proposed. The method evaluated the protection provided by the structure from the perspective of an occupant of the structure. The method incorporated both structural and non-structural properties of the building and expressed the protection offered by the structure in terms of the fraction of expected fatalities if the building were hit by a hurricane.

The objective of this report is to demonstrate the use of the methodology on an existing structure that may function as a potential vertical shelter. In the context of this study, a structural risk analysis has at least three goals: a) to estimate the expected fraction of the inhabitants that would be killed if a hurricane of a given intensity were to hit the structure, b) to provide some measure of the uncertainty surrounding the latter estimate, and c) to estimate the cost of upgrading the structure to satisfy certain predetermined functional/safety levels. As shown in Table 1, the probability of nineteen events must be provided as input to the model. Here we will demonstrate how estimates of events X_2, X_5, X_8, X_{11}, X_{14}, and X_{19} are obtained.

Some General Comments

The proposed procedure relies heavily on classical structural reliability methods. Here we make frequent use of the so called First-Order Second-Moment methods. This method uses the mean (first moment) and the variance (second moment) of the load on a structure and resistances of the structure to estimate the probability of failure. For example, let random variable R be the resistance (strength) of a structural element and random variable S be the load on the element, then random safety margin, Z, is given by:

$$Z = R - S \tag{1}$$

If $Z \leq 0$ the structural element has failed. If $Z > 0$ the element is safe. the measure of safety is the probability of failure, i.e.,

$$P_f = P[Z \leq 0] \tag{2}$$

The probability of failure may be estimated from the mean and variance of Z:

$$\begin{aligned} E[Z] &= E[R] - E[S] \\ Var[Z] &= Var\ [R] + Var\ [S] \\ \beta &= E[Z]/\sqrt{Var[Z]} \\ P_f &= \Phi(-\beta) \end{aligned} \tag{3}$$

where β is commonly called the Reliability Index and Φ is the standard normal distribution function.

In general, the safety margin is a function of several resistance and load variables and may be written:

$$Z = g(x_1, x_2, x_3, \dots, x_n) \tag{4}$$

In such cases, the probability of failure is evaluated using so called mean value methods, and advanced first order second moment methods (3). In the ensuing example, mean value methods will be used. In these methods, the first and second moments of $Z = g(x_1, x_2, x_3, \ldots, x_n)$ are evaluated using the results:

$$E[Z] = g(E[X_1], E[X_2], \ldots, E[X_n])$$
$$\mathrm{Var}[Z] = \sum_{i=1}^{N} (\partial g/\partial X_i)^2 \mathrm{Var}[X_i] \tag{5}$$

where the random variables X_i are assumed to be statistically independent.

Description of the Structure

The structure is located on the southern side of Galveston Island and serves as a hotel. The structure, built in 1983, covers a floor plan area of approximately 16,000 square feet and has seven stories. The height of the first story is 10.3 feet above ground level and the height of each succeeding floor is 8.7 feet. The building codes governing the design include the Standard Building Code (1979), the AISC Specifications for the Design, Fabrication, and Erection of Structural Steel For Buildings (1978) and ACI 318-77.

The terrain to the north, east, and west of the structure consists primarily of low-rise houses and trees. The Gulf of Mexico faces south. Fifty feet to the east is a building under construction. Three hundred feet to the north are a row of 3-story apartment buildings. One hundred and fifty feet to the west is a 20-story structure. About 200 feet to the south is the Seawall.

Figure 1 shows two elevations and a typical floor plan of the structure. The foundation consists of 2'-3" deep reinforced concrete grade beams supported on 16-in diameter concrete friction piles. The safe load capacity of each pile is specified at 60 tons.

The structural material of the superstructure is concrete. Lateral loads are resisted by four cores - two elevator shafts and two stairwells - and thirteen pre-cast, post-tensioned concrete shear walls. The shear walls also support the floors which consist of pre-cast, pre-stressed planks. Vertical post-tensioning rods in the cores and shear walls are post-tensioned to 105,000 psi. The shear walls are attached to the grade beam via post-tensioning and grouting, and the floor planks bear on a shoulder in the shear walls and are grouted in place. The roofing support is the same as the other floors but is covered with a 3-inch insulation board over which is applied a 4-ply roofing membrane. Wall A, which faces the gulf, consists of 168 window wall units. Each unit is comprised of 1/4" float glass in bronze anodized frames. Walls B and D consist of the end shear wall with a brick veneer. Wall C is made up of a metal stud wall with a stucco finish.

Analysis of Frame Failure

The definition of frame failure for the example structure subjected to a hurricane is shown in Figure 2. Since failure of any of the four cores will lead to frame failure and each core can fail in four ways, sixteen failure functions for the cores must be defined. Similarly, since failure of any of the seven shear walls constitutes failure and each shear wall can fail in 14 ways (seven floors with two loading conditions), 98 failure functions for the shear wall system must be defined. Thus, to estimate the failure probability of the frame we considered a total of 114 failure functions. The failure functions used in this analysis are summarized in Tables 2 and 3. The supplementary data needed to write the equations have been obtained from the plans. In Table 2, note that:

σ_{uc} ≡ the ultimate strength for concrete

σ_{up} ≡ the ultimate strength for the post-tensioning rods

I ≡ second moment of area for the core section

V_i ≡ the wind velocity of category i hurricane

W_i ≡ the distributed loading for a category i hurricane

Note also that the equivalent distributed load for a category i hurricane (i = 1, 2, 3, 4, 5) is estimated by the equation:

$$W_i = P_i l \qquad (6)$$

where P_i is the pressure acting on the structure in a category i hurricane, and l is the width over which the load acts. For this analysis we have estimated P_i using the equation:

$$P_i = 0.00256 C_D V_i^2 \qquad (7)$$

where C_D is the shape factor for the building. The statistics for the various hurricanes and the resulting loading on the example structure are summarized in Table 4. The upper limit for a Category 5 hurricane is taken to be 175 MPH.

In Table 3, T_R is the shear strength of the concrete, σ_p is the compressive stress due to the prestress, σ_L is the compressive stress due to the dead and live load acting on the floor, and A and C_i are, respectively, the cross-sectional area of the shear wall and the length over which the load W_i is acting.

The statistics of the basic variables used to estimate the failure probability for the cores and the shear walls are summarized in Tables 5 and 6. These values were obtained on the basis of recommendations [see Reference (3)] and data provided on the plans.

The results for the failure probabilities for the cores and shear walls are summarized in Tables 7 and 8. These values correspond to the inputs needed for the bottom row of boxes in Figure 2. Once these values are known, the probability of the event that the frame fails may be calculated directly.

Analysis of Roof Failure

The definition of roof failure for the example structure subjected to hurricane force winds is shown in Figure 3. The roof is made up of a collection of planks each of which is connected to the shear walls as shown in Figure 4. Under extreme wind loading a single panel may fail in one of several ways. First, the panel may fail as a result of excessive tensile stresses in the top fibers. Second, bolts denoted by an "A" may fail in tension or the anchorage in the plank may fail by pulling out. Third, the bolts denoted by "B" may fail in shear or the anchorage in the shear wall may fail by shear pull-out. If one plank should fail, the roof as a system may still function satisfactorily. However, if this percentage were to increase, the protection offered by the roof becomes questionable. Here a loss of more than 5% of the roofing units is defined to constitute failure.

The failure function for these limit states were developed using the rationale:

$$\text{Safety Margin} = \text{Resisting Capacity} - (\text{Uplift Effect} - \text{Dead Load Effect}) \qquad (8)$$

Representative failure functions for the five failure modes are listed in Table 9. The equations are based inpart on discussions presented in References (2) and (1). The values used for the resistance parameters are listed in Table 10. The statistics for the uplift forces due to the wind, assuming maximum uplift on the roof, are provided in Table 11. The reliability index for each failure mode and the corresponding failure probability is given in Table 12.

The results for the failure of the roof as a system is provided in Table 13. The results in Table 13 were obtained using the results in Table 12 and the binomial probability distribution. The final probability that the roof system fails can now be obtained using the logic in Figure 3.

Analysis of Window Failure

The wall facing the gulf is fitted with 168 window units. Each unit is 8.8 ft. wide and 9.0 ft high. The unit is supported by the shear wall on one side and a partition on the other side. The top and bottom supports are provided by the respective floors. The typical unit also contains a door and houses an air conditioning unit. The glass used is 1/4" float glass mounted in a bronze anodized frame.

This unit may fail if a) the glass breaks, b) the door fails, or c) the connection between the unit and the supports fail. However, since information on the design details for the units are not available a more direct approach to the reliability evaluation of the wall system is taken here. First, we will define failure of the windows to be the event that at least 10 percent of all window units fail. Second, the failure of a single panel is defined to be the event that the hurricane forces exceed the unit's resistance. The unit's resistance is exceeded if water and wave forces exceed the resistance, or if a projectile impact results in failure.

The failure function for a window unit may be given by:

$$Z_i = R - S_i \tag{9}$$

where R is the resistance of the window unit, S_i is the load on the unit subjected to a hurricane of category i, and Z_i is the safety margin. Let R be represented in units of pressure. In this analysis R is estimated as follows: a) from the engineering drawings, the cladding has been designed to resist a net pressure of 52 psf. Assuming a safety factor of 1.1, the nominal resistance of the system $R_n = 52(1.1) = 57.2$ psf Furthermore, assume that:

$$\bar{R}/R_n = 1.05 \quad \text{and} \quad COV[R] = 0.15 \tag{10}$$

[see Reference (3)]. The mean and standard deviation for the resistance of a single panel becomes:

$$\bar{R} = 60.06 \text{ psf}, \ \sigma_R = 9.01 \text{ psf} \tag{11}$$

Using previously obtained values for S_i, the failure characteristics for the panels and the system is summarized in Table 14. Again the system response was obtained by utilizing the binomial distribution in which the probability of failure for a single unit becomes the probability of success in the binomial distribution.

Analysis of Stud Wall Failure

The north wall of the structure comprises primarily of 3 5/8" metal studs 16" O.C. The analysis of this wall will proceed analogously to that of the window units. The wall system enclosing one room (9.0 ft X 8.125 ft.) is considered one unit. Failure occurs when the hurricane forces exceed the design strength. The failure function then becomes:

$$Z_w = R_w - S_i \qquad (12)$$

Here we set $\bar{R}_w$ = 60.06 psf and σ_R = 6.01 psf, using the assumption that the cladding has been designed to resist a pressure of 52 psf and setting the coefficient of variation of the stud wall to be 0.1.

The results summarizing the safety of the stud wall are summarized in Table 15.

Note that the failure hazard of the exterior east and west walls of the structure were ignored since the walls consist of a brick veneer on the concrete shear wall.

Consequences of Failure

The consequences of failure of any of the building components depend upon many factors. First, they depend on the nature of the construction materials. For example, if failure occurs in a brittle manner the consequences of failure would be more grave than if the construction material failed in a more ductile manner. Second, the consequences also depend upon the disposition of the sheltered population. For example, immobile or elderly evacuees may not be able to out-maneuver the hazards posed by a failing structure to the same degree as would a group of younger or more agile occupants. Thus, the consequences of failure of the structural protection should incorporate these considerations. However, on the other hand, since the characteristics of the sheltered occupants are not known beforehand, it seems reasonable to assume the worst case scenario and assign the levels of consequences accordingly. This assumption results in assigning the consequences of structural collapse to frame failure, foundation failure, roof failure, and cladding failure. Since no organized data is available for hurricanes, the data that does exist for earthquakes - although it leaves much to be desired - is assumed to approximate the consequences of the same damage state that would occur in a hurricane. For the structure under consideration - a reinforced concrete structure greater than five stories high - the mean fraction of fatalities is 0.6 and the standard deviation is 0.44 (5). These numbers were fitted to a log-normal distribution to generate the data needed for the model.

Risk of Fatalities

The data developed in the previous sections were used as input for the model described in Reference (4), to estimate the risk of utilizing the example structure in various hurricanes. The structural input and the results are summarized in Tables 16 and 17.

Plots of the probability of failure of the various subcomponents versus the magnitude of the hurricane are shown in Figure 5. Note that the probability of frame failure - as compared with cladding and roof failure - is relatively insensitive to the magnitude of the hurricane. If one closely examines Tables 16 and 17, and Figure 5, it will become apparent that the expected fatalities associated with Category 1 and Category 2 hurricanes are due primarily to the chance of frame failure. On the other hand, the expected fatalities for Categories 3, 4, and 5 result primarily from roof and cladding failure. Since the structural frame of no engineered multi-story structure has been observed to collapse in a Category 1 or 2 hurricane, the occurrence of that event may be ignored. Motivated by these arguments, if frame failure is ignored for the entire calculation, the results shown in Table 18 are obtained. Note that the only difference between the results in Tables 18 and 19 is that, in the latter case, no fatalities are predicted for a Category 1 or 2 hurricane.

Cost to Upgrade

These results show that the risks associated with using the structures in a Category 3 or greater hurricane are quite substantial. The analysis indicates that in order to lessen the risk of a fatality, upgrading efforts should be directed at the window walls, the stud walls, and the roof. Furthermore, the failure analysis of the roof indicates that the shear beam-plank connections experiencing tension (while resisting uplift forces) should be reinforced.

To estimate the cost to upgrade the example structure such that the expected fatalities for a Category 3 hurricane are approximately those for the Category 1 and 2 storms, we may determine the cost of the following renovations: a) upgrade the window walls to resist a design pressure of 60 psf, b) upgrade the stud walls to resist a design pressure of 60 psf, and c) upgrade the roof to resist an uplift pressure of 88 psf. Note that these numbers correspond to the maximum wind forces exerted on the cladding and the roof by a Category 3 hurricane. One way of meeting these requirements is to a) replace the window-wall units and install 1/2" tempered glass, b) replace the existing stud wall system, and c) add an additional tension bolt to each roof plank.

The upgraded reliabilities (ignoring the consequences of frame failure) for the structure using these changes are shown in Table 19. The expected fatalities are shown in Table 20. Note that the upgrading effected only the risk level associated with the Category 3 hurricane, the results for all others remained essentially unchanged.

A summary of the cost to upgrade the structure is presented in Table 21. The estimated cost to upgrade, excluding profits, is $600,000.00. In terms of cost to upgrade per square foot this result may be alternatively stated as follows: a) the cost to upgrade the window wall is $42 per square foot, b) the cost to upgrade the stud wall is $15 per square foot, and c) the cost to upgrade the roof is $0.57 per square foot. Note that the respective areas of the south wall, north wall, and roof are 18,296 sq ft, 18,296 sq ft, and 16,000 sq ft.

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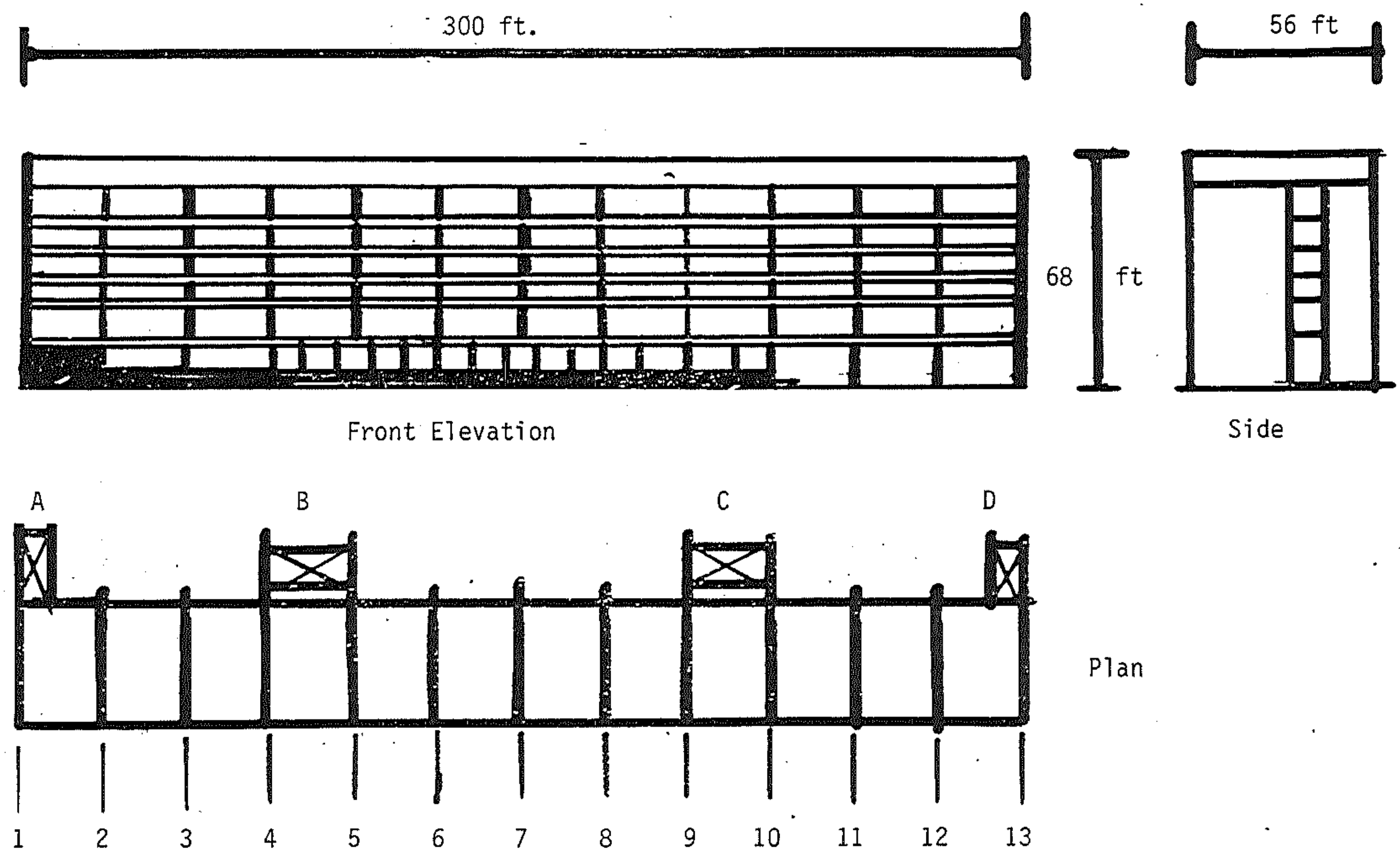

Figure 1. Plan and Elevation of Example Structure

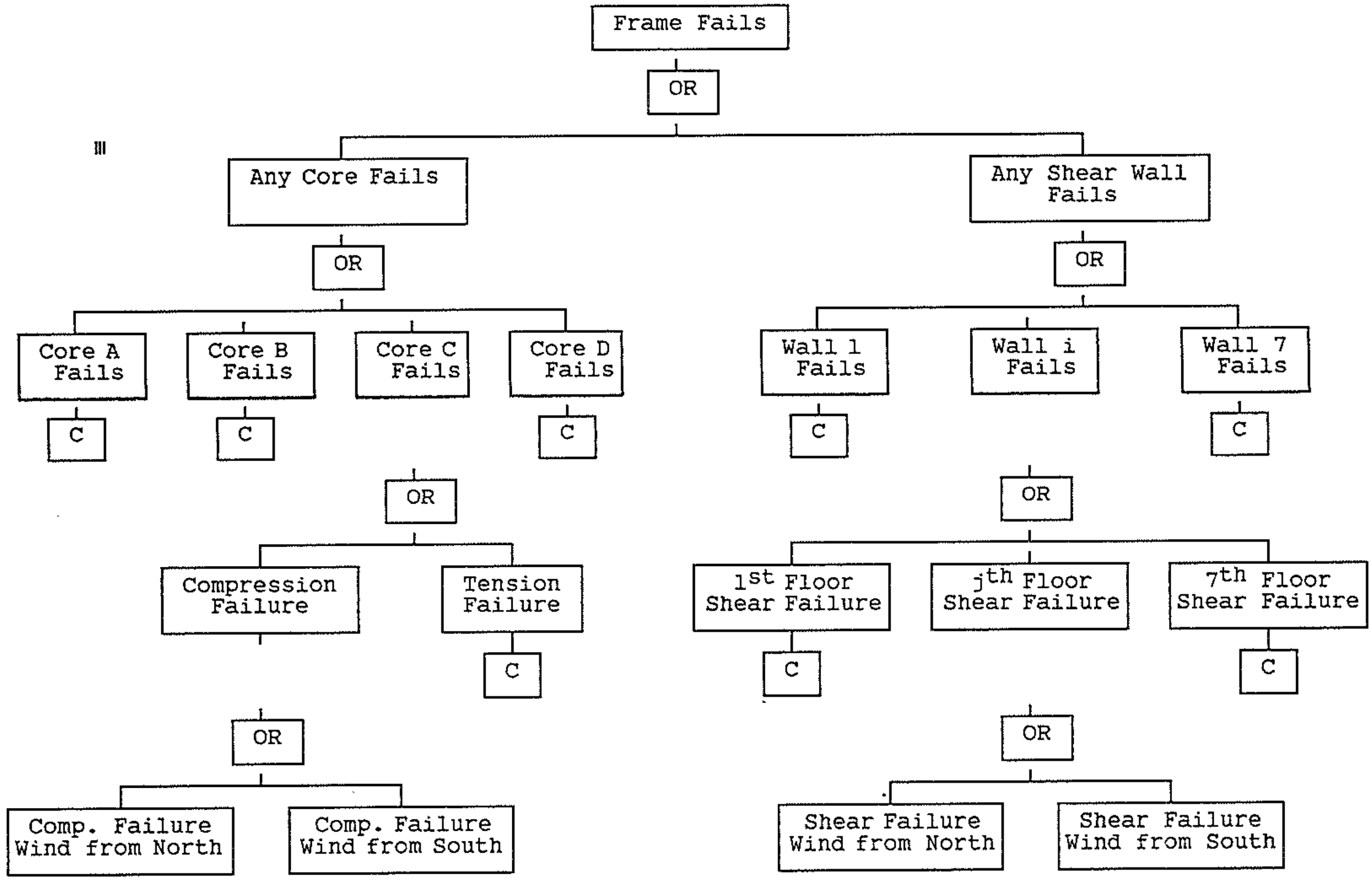

Figure 2. Definition of Frame Failure for Example Structure

C Continuation

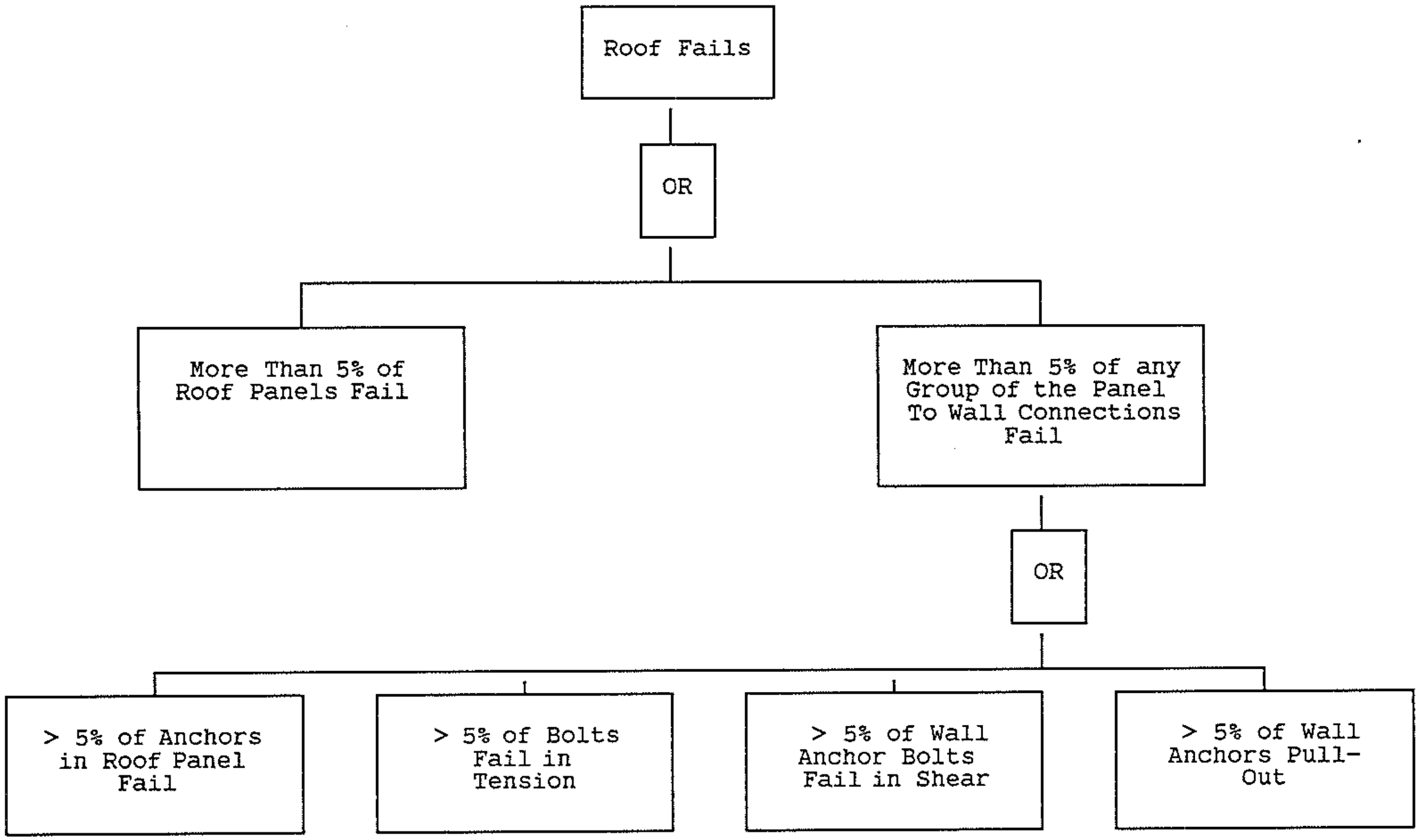

Figure 3. Definition of Failure for Roof

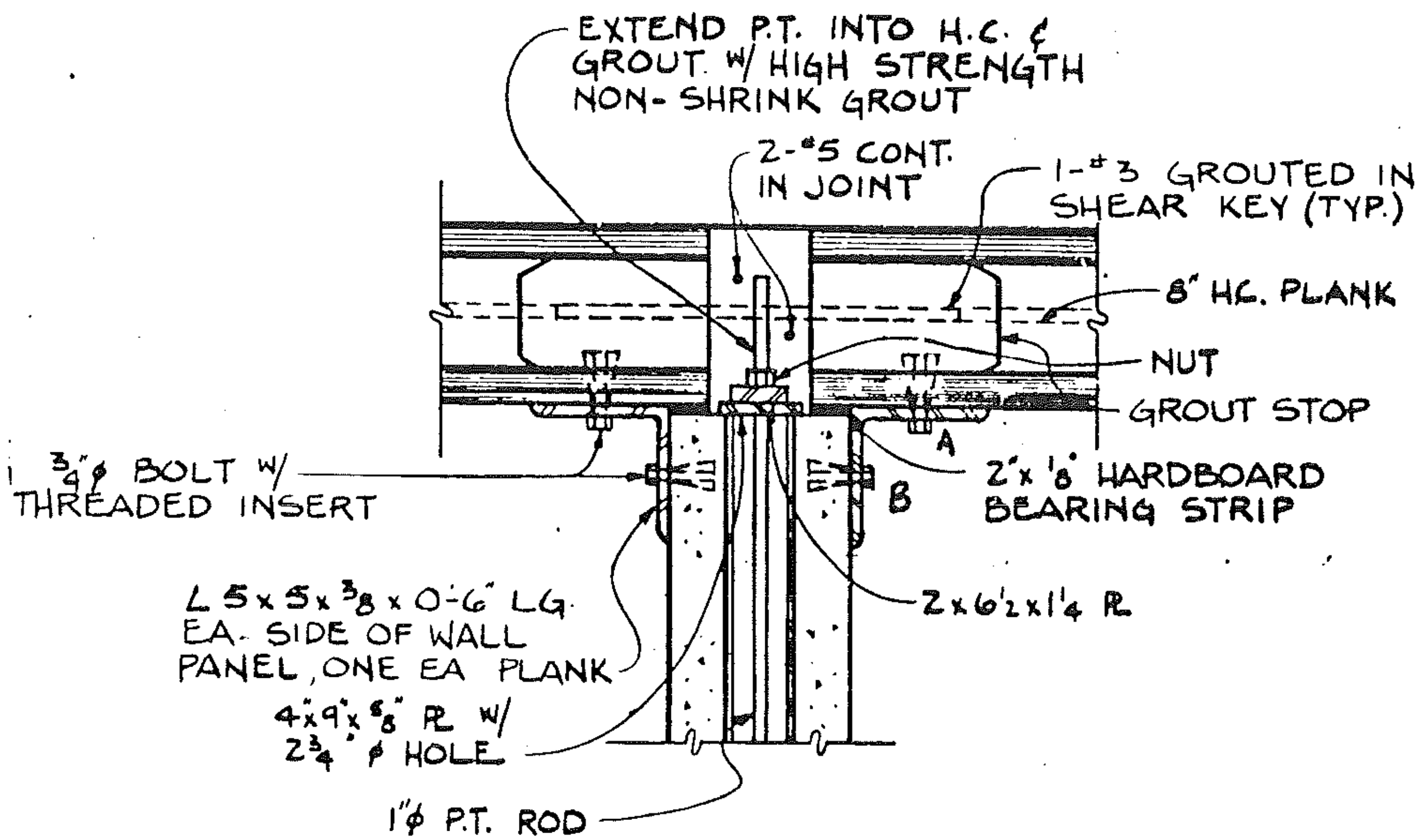

Figure 4. Connection Details for a Typical Roof Panel

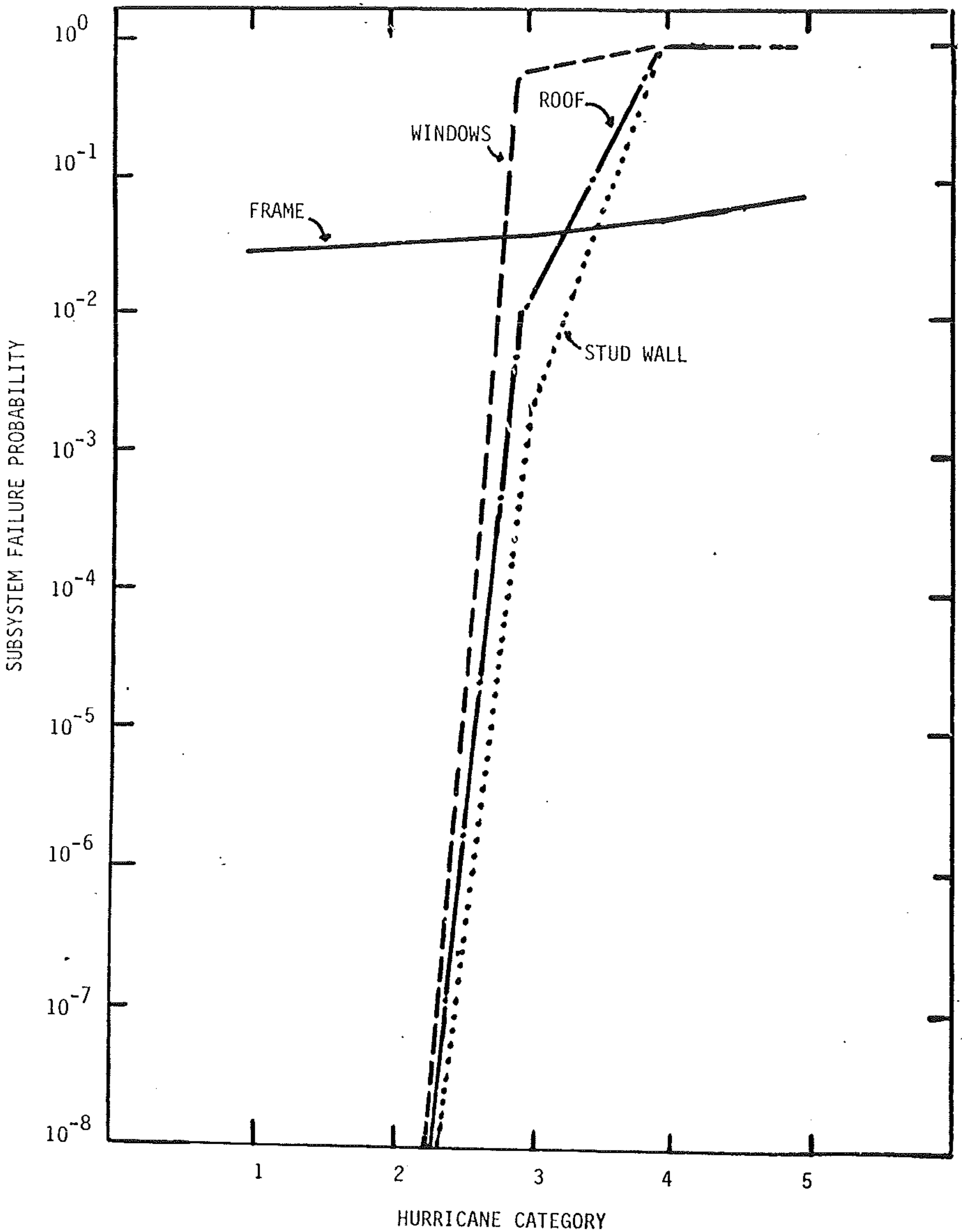

Figure 5. Sensitivities of Subsystem Failure

Table 1. Summary of Basic Events

Event	Description of Event
X_1	Wind hazard occurs
X_2	Wind forces exceed frame strength
X_3	Person is exposed/frame fails
X_4	Injury is fatal/frame fails
X_5	foundation fails
X_6	Person is exposed/foundation fails
X_7	Injury is fatal/foundation fails
X_8	Wind forces exceed roof strength
X_9	Person is exposed/roof fails
X_{10}	Injury is fatal/roof fails
X_{11}	Wind forces exceed cladding resistance
X_{12}	Person is exposed/cladding fails
X_{13}	Injury is fatal/cladding fails
X_{14}	Wind forces exceed opening resistance
X_{15}	Person is exposed/opening fails
X_{16}	Injury is fatal/opening fails
X_{17}	Wind forces exceed int. part. resistance
X_{18}	Person is exposed/partition fails
X_{19}	Injury is fatal/partition fails

Table 2. Failure Functions

Description of Failure Mode	Safety Margin (Failure Function)
Concrete in elevator core fails by crushing (Wind from south)	$Z_1 = \sigma_{uc} - [17{,}299.2(C/I)W_i + \sigma_{ps} + 54.2]$
SAME (Wind from north)	"
Tendons in elevator roof fail in tension (Wind from south)	$Z_2 = \sigma_{up} - [\sigma_{ps} + 17{,}299.2(C/I)W_i]$
SAME (Wind from north)	"
Concrete in stairwell core fails by crushing (Wind from south)	$Z_3 = \sigma_{uc} - [17{,}299.2(C/I)W_i + \sigma_{ps} + 54.2]$
SAME (Wind from north)	"
Tendons in stairwell core fail in tension (Wind from south)	$Z_4 = \sigma_{up} - [\sigma_{ps} + 17{,}299.2(C/I)W_i]$
SAME (Wind from north)	"

Table 3. Failure Functions for a Typical Shear Wall

Description of Failure Mode	Safety Margin (Failure Function)
Concrete fails in shear in i^{th} floor	$Z_i = T_R - \{(\sigma_p + \sigma_i)^2 + 4(1.2C_iW_i/A)^2\}^{1/2}/2$

Table 4. Hurricane Categories and Their Resulting Loading on the Structure

Hurricane Category (i)	Parameters					
	Wind Speed V_i (MPH)		Wind Pressure P_i (psf)		Live Load W_i (plf)	
	mean	variance	mean	variance	mean	variance
1	84.5	12.25	23.76	9.52	23.76l	9.52l^2
2	103.0	5.44	35.31	15.02	35.32l	15.02l^2
3	120.5	10.03	48.32	29.80	48.32l	29.80l^2
4	143.0	16.00	68.05	60.81	68.05l	60.81l^2
5	165.5	10.03	91.15	95.26	91.15l	95.26l^2

Table 5. Statistics of Resistance Variables for Cores

Variable	Symbol	Units	Mean	COV	Assumed Distribution
Comp. strength of concrete	σ_{uc}	lb/in^2	4,200	0.10	Normal
Ultimate strength of tendons	σ_{up}	lb/in^2	157,500	0.10	Normal
Stress in tendons due to prestress	σ_{ps}	lb/in^2	105,000	0.10	Normal
Stress in conc. due to prestress	σ_{psc}	lb/in^2	Cores:		
			B&C:138.9	0.10	Normal
			A&D:120.8	0.10	Normal
Moment of inertia	I	in^4			
B&C			19,645,863	0.05	Normal
A&D			19,083,340	0.05	Normal

Table 6. Statistics of Basic Variables for Shear Wall

Variable Number	Symbol	Units	Mean	Coefficient of Variation
Shear resistance of concrete	T_R	KSF	91.07	0.10
Prestress on concrete	σ_p	KSF	28.7	0.00
Category 1 load intensity	W_1	KLF	0.594	0.10
Category 2 load intensity	W_2	KLF	0.883	0.07
Category 3 load intensity	W^3	KLF	1.208	0.08
Category 4 load intensity	W_4	KLF	1.702	0.08
Category 5 load intensity	W_5	KLF	2.293	0.07

Table 7. Results from the Analysis of the Cores

Failure Mode	Reliability Indices (Failure Probabilities)				
	Hurricane Category				
	1	2	3	4	5
Elevator cores fail in compression (wind from south)	8.14 (*)	7.83 (*)	6.84 (*)	5.66 (*)	4.84 (*)
Elevator cores fail in compression (wind from north)	6.98 (*)	6.53 (*)	5.18 (1.11×10^{-7})	3.87 (5.44×10^{-5})	3.08 (1.04×10^{-3})
Elevator cores fail in tension (wind from south)	2.61 (4.53×10^{-3})	2.55 (5.39×10^{-3})	2.44 (7.34×10^{-3})	2.28 (1.13×10^{-2})	2.11 (1.74×10^{-2})
Elevator cores fail in tension (wind from north)	2.64 (4.15×10^{-3})	2.58 (4.94×10^{-3})	2.50 (6.21×10^{-3})	2.38 (8.66×10^{-3})	2.24 (1.25×10^{-2})
Stairwell cores fail in compression (wind from south)	9.34 (*)	9.15 (*)	8.52 (*)	7.64 (*)	6.94 (*)
Stairwell cores fail in compression (wind from north)	9.42 (*)	9.24 (*)	8.68 (*)	7.86 (*)	7.21 (*)
Stairwell cores fail in tension (wind from south)	2.72 (3.26×10^{-3})	2.70 (3.47×10^{-3})	2.67 (3.79×10^{-3})	2.62 (4.40×10^{-3})	2.58 (4.94×10^{-3})
Stairwell cores fail in tension (wind from north)	2.73 (3.17×10^{-3})	2.71 (3.36×10^{-3})	2.69 (3.57×10^{-3})	2.65 (4.02×10^{-3})	2.61 (4.53×10^{-3})

*Probability of Failure $< 10^{-7}$.

Table 8. Results from the Analysis of Typical Shear Wall

Failure Mode (wind from north or south)	Reliability Indices (Failure Probabilities) Hurricane Category 1	2	3	4	5
First floor fails in shear	6.01 (*)	5.95 (*)	5.87 (*)	5.68 (*)	5.37 (*)
Second floor fails in shear	6.24 (*)	6.17 (*)	6.04 (*)	5.78 (*)	5.35 (*))
Third floor fails in shear	6.63 (*)	6.57 (*)	6.46 (*)	6.23 (*)	5.85 (*)
Fourth floor fails in shear	7.06 (*)	7.01 (*)	6.93 (*)	6.76 (*)	6.47 (*))
Fifth floor fails in shear	7.50 (*)	7.47 (*)	7.41 (*)	7.29 (*)	7.10 (*)
Sixth floor fails in shear	7.93 (*)	7.91 (*)	7.88 (*)	7.81 (*)	7.71 (*)
Seventh floor fails in shear	8.39 (*)	8.39 (*)	8.39 (*)	8.39 (*)	8.38 (*)

*Probability of Failure < 10^{-7}.

Table 9. Failure Functions for Roof Element

Description of Failure Mode	Safety Margin (Failure Function)
Tension failure of roof bolt	$Z = T_{ult} - [40{,}901W_i - 2454]$
Panel fails	$Z = M_u - [M_{(W+DL+LL)i} - 2454]$
Shear failure of roof bolt	$Z = V_{ult} - [40{,}901W_i - 2454]$
Tension pull-out in roof panel	$Z = 6.7\ \sqrt{\sigma_{uc}} - [40{,}901W_i - 2454]$
Concrete shear failure at wall bolt	$Z = 97.15\sigma_{uc}^{1/2} - [40{,}901W_i - 2454]$

Table 10. Parameters used to Evaluate Safety of Roof

Variable	Units	Mean	COV	Distribution
Compressive strength of concrete (σ_{uc})	psi	4,200	0.10	Normal
Ultimate strength of roof bolt in tension, (T_{ult})	lb	26,400	0.10	Normal
Ultimate strength of roof bolt in shear, (V_{ult})	lb	15,300	0.10	Normal

Table 11. Uplift Pressures on A Typical Roof Panel

Hurricane Category	Uplift Pressure* on Typical Plank (PSF)	
	μ_p	σ_p
1	29.25	5.11
2	43.45	5.52
3	59.47	9.69
4	83.76	11.20
5	112.19	13.46

*$P = 0.00256 C_D V^2$, $\mu_{CD} = -1.6$, $COV[C_D] = 0.1$

Table 12. Reliability Indices for Panel Failure

Failure Mode	Reliability Indices (Failure Probabilities)				
	Hurricane Category				
	1	2	3	4	5
Panel failure	14.04 (*)	13.66 (*)	13.23 (*)	12.58 (*)	11.82 (*)
Tension failure of roof bolt	10.47 (*)	10.24 (*)	9.97 (*)	9.58 (*)	9.13 (*)
Shear failure of roof bolt	10.79 (*)	10.40 (*)	9.90 (*)	9.23 (*)	8.43 (*)
Tension pull-out in roof panel	13.71 (*)	8.33 (*)	1.94 (2.62×10^{-2})	-2.06 (1.96×10^{-2})	-5.36 (1.00)
Shear failure in shear wall	22.40 (*)	20.46 (*)	16.23 (*)	12.95 (*)	9.31 (*)

*Probability of Failure < 10^{-7}.

Table 13. Failure Probabilities for Roof as a System

Failure Mode	Failure Probabilities				
	Hurricane Category				
	1	2	3	4	5
> 5% of roof panels fail	*	*	*	*	*
> 5% off anchorages in roof panel fail	*	*	*	*	*
> 5% of bolts in panel fail	*	*	$1.14X10^{-2}$	1.00	1.00
> 5% of shear wall anchorages fail	*	*	*	*	*
> 5% of bolts fail in shear	*	*	*	*	*

*Negligible < 10^{-7}

Table 14. Summary of Failure of Characteristics of Window Units

Failure Mode	Reliability Indices (Failure Probabilities)				
	Hurricane Category				
	1	2	3	4	5
Unit fails	3.99 (5.01×10^{-5})	2.64 (4.14×10^{-3})	1.20 (1.15×10^{-1})	-0.76 (7.76×10^{-1})	-2.86 (9.98×10^{-1})
> 10% of units fail	(*)	(*)	(7.62×10^{-1})	(1.00)	(1.00)

*Probability of failure $< 10^{-7}$.

Table 15. Summary of Failure Characteristics of Stud Wall

Failure Mode	Reliability Indices (Failure Probabilities)				
	Hurricane Category				
	1	2	3	4	5
Unit fails	5.61 (*)	3.78 (7.84×10^{-5})	1.65 (4.95×10^{-2})	-0.98 (8.36×10^{-1})	-3.59 (9.99×10^{-1})
> 10% of units fail	(*)	(*)	(1.79×10^{-3})	(1.00)	(1.00)

*Probability of failure $< 10^{-7}$.

Table 16. Structural Input to Risk Model for Example Structure

Failure Mode	Failure Probabilities				
	Hurricane Category				
	1	2	3	4	5
Frame fails	3.00×10^{-2}	3.45×10^{-2}	4.20×10^{-2}	5.72×10^{-2}	8.20×10^{-2}
Foundation fails	*	*	*	*	*
Windows fail	*	*	6.62×10^{-1}	1.00	1.00
Stud walls fail	*	*	1.70×10^{-3}	1.00	1.00
Roof fails	*	*	1.14×10^{-2}	1.00	1.00

*Indicates that the value is assumed to be or is less than 10^{-7}.

Table 17. Risk of Using Example Structure in Various Hurricanes

Hurricane Category	Expected Fraction of Fatalities	Standard Deviation of Expected Fatalities
1	0.014	0.09
2	0.016	0.10
3	0.31	0.33
4	0.72	0.26
5	0.72	0.26

Table 18. Risk of Using Example Structure But Ignoring Frame Failure

Hurricane Category	Expected Fraction of Fatalities	Standard Deviation of Expected Fatalities
1	0.00	0.00
2	0.00	0.00
3	0.30	0.33
4	0.72	0.26
5	0.72	0.26

Table 19. Structural Input to Risk Model for Upgraded Example Structure

Failure Mode	Failure Probabilities				
	Hurricane Category				
	1	2	3	4	5
Frame fails	*	*	*	*	*
Foundation fails	*	*	*	*	*
Windows fail	*	*	*	1.00	1.00
Stud walls fail	*	*	*	1.00	1.00
Roof fails	*	*	*	1.00	1.00

*Indicates that the value is assumed to be or is less than 10^{-7}.

Table 20. Risk of Using Upgraded Structure in Various Hurricanes

Hurricane Category	Expected Fraction of Fatalities	Standard Deviation of Expected Fatalities
1	0.00	0.00
2	0.00	0.00
3	0.00	0.00
4	0.72	0.26
5	0.72	0.26

Table 21. Summary of Cost to Upgrade Example Structure

Item	Pay Unit	Estimated Quantity	Unit Cost	Estimated Cost
1 3/4" 16 GA door	Each	168	$ 300.00	$ 50,400.00
63 SF steel frame	Each	168	1,827.00	306,936.00
1/2" tempered plate glass	Each	168	504.00	84,672.00
Demolition of existing frame	Each	168	20.00	3,360.00
Demolition of existing 63 SF stud wall	Each	168	40.00	6,720.00
63 SF-3 5/8" studs 16 GA	Each	168	100.00	16,800.00
1 3/4" 16 GA steel ribbed core	Each	168	300.00	50,400.00
63 SF dry-wall	Each	168	55.00	9,240.00
63 SF stucco	Each	168	440.00	73,920.00
Installation of tension bolts	Each	240	38.00	9,120.00
Total Engineer's Estimate				$611,568.00