Title: **Multihazard Design of Tall Buildings**

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Multihazard Design of Tall Buildings

This presentation is based on a paper by the presenter, Joong C. Lee, and Mohammed M. Ettouney.

Hazards that might affect tall buildings include wind, seismic, and as of late, blast. It is recognized that such an occurrence constitutes an abnormal event in likelihood, building response, and impact. Because of the necessity of mitigating such an event and the accompanied costs, the role of multihazard design considerations and their potential cost reduction are gaining importance. Multihazard design considerations imply the utilization of design measures for abnormal events (earthquakes, wind, blast, progressive collapse, etc.) to accommodate design demands of other abnormal events. By doing so, the total costs of design mitigation measures would be reduced yet satisfy all structural safety needs.

The two main obstacles in applying a multihazard approach to tall buildings are: 1) abnormal conditions vary immensely in almost all aspects of design and analysis, and 2) analyzing a tall building is a daunting task. In addition, all known design guidelines, standards, and codes have not followed multihazard strategies. These factors, and others, make it extremely difficult to qualitatively and quantitatively accommodate a true multihazard design (maintaining safety while reducing cost for all pertinent hazards).

This work presents an integrated approach for multihazard design for tall buildings. The approach utilizes important aspects of hazard assessment as the main linkage between different hazards, in conjunction with current design guides, standards, and codes. The proposed method has the potential to quantify multihazard design of tall buildings for the first time.

Multihazard Design of Tall Buildings

By

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ABSTRACT

Hazards that might affect tall buildings include wind, seismic and lately, blast. It is recognized that such an occurrence constitutes an abnormal event in likelihood, building response and impact. Because of the necessity of mitigating such an event as well as the accompanied costs of such mitigation, the role of multihazard design considerations and their potential cost reduction are gaining importance. Multihazard design considerations imply the utilization of design measures for other abnormal events (earthquakes, wind, blast, progressive collapse, etc.) to accommodate some of the design demands of other abnormal events. By doing so, the total costs of design mitigation measures would be reduced, while satisfying all structural safety needs.

This work presents several applications of multihazard considerations for tall buildings. Three realistic examples are given. The first example explores the interaction of building resiliencies as it resists different hazards (wind and progressive collapse). Another example shows that the efficiencies of different structural systems (EBF vs. CBF) can vary, depending on the types of multihazards considered. Finally, it is shown that when hazards conflict in their demands on the building, the effects on life-cycle costs can be large. In all, the cost implications of including or ignoring multihazards during design phase are shown to be in the range of 1.5%-18%. The proposed methods and examples show that quantifying multihazard design of tall buildings must be performed during the design phase in order to save unnecessary expenditures.

Keywords

Multihazard, blast loading, earthquake, wind, progressive collapse.

INTRODUCTION

The two main obstacles in applying a multihazard approach to tall buildings are: 1) abnormal conditions vary immensely in almost all aspects of design and analysis and 2) analyzing a tall building is, by default, a daunting task. In addition, all known design guidelines, standards and codes have not followed multihazard strategies. These factors, and others, have made it extremely difficult to accommodate a true multihazard (maintain safety while reducing cost for all pertinent hazards) design, qualitatively and quantitatively.

In this paper we explore several aspects of multihazard designs of buildings. We follow some of the principles of the Theory of Multihazards that was introduced by Ettouney, et. al. (2005). The theory argued for the inherent resiliency of any structural system to resist all type of hazards. The first example in this study is the inherent resiliency for wind and progressive collapse of a tall building. Next, the varied efficiency of structural systems according to the types of hazards (single-, or multi-) is studied. Finally the life-cycle cost of hazards, and the effects of multihazard considerations is shown in an example. It is shown that the cost effects of ignoring multihazard demands can be as large as 18%.

Inherent Multihazards Resiliency: Wind and Progressive Collapse Example

Consider the situation of a tall building that is located in an area without any seismic provision requirement. Only wind loading is required for the design of the building. However, the building is also

required to follow measures for progressive collapse mitigation. There are two procedures for designing such a situation: a one-hazard-at-a-time procedure, and a multihazard procedure. The one-hazard-at-a-time procedure will design the building for wind and progressive collapse independently. First: a wind design will be evaluated; secondly, the measures for progressive collapse demands will be applied. The second approach, a multihazard approach, will design for wind first, then apply progressive collapse measures, then re-adjust the wind designs. The differences between the two approaches are illustrated below.

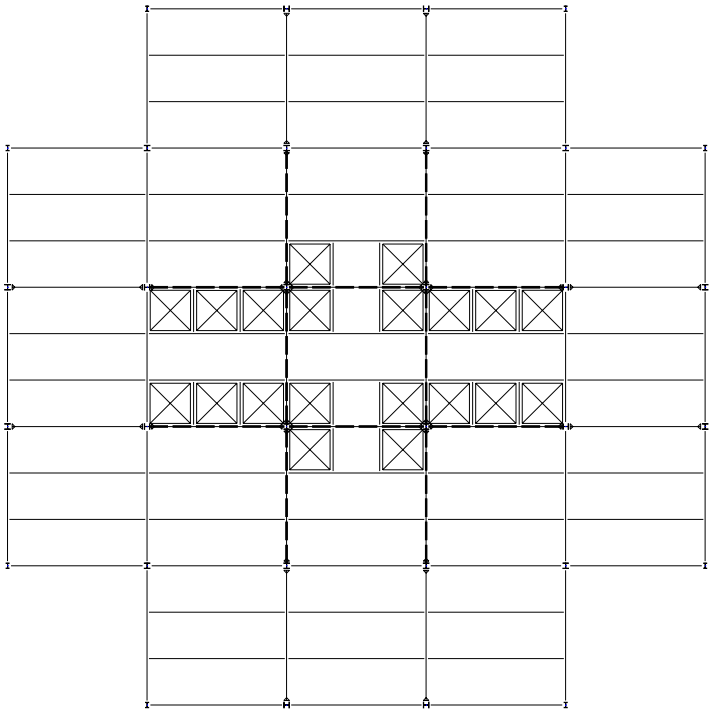

Figure 1 – Floor Layout of Building A

To compare the two above design procedures for wind and progressive collapse hazards, a test case building was created with the intent of closely simulating possible real world design. At the same time, the design was kept as simple as possible to improve the efficiency of effort computations, without sacrificing the accuracy of results. The building will be referred to as Building A. The building is 49 stories, with a typical floor-to-floor height of 12'-6" and a double-height first floor, giving a total building height of 625'.

The typical building framing plan is shown in Fig. 1. It is a simple 5-bay by 5-bay plan with columns spaced at 30 feet, giving plan dimensions of 150'x150'. The corner bays are removed to create a cruciform shape in plan. The elevator core mimics the shape of the building with a pair of central corridors that cross at the center. Two braced frames in each orthogonal lie on the edges of the edges of the core, keeping with the cross theme of the design.

Fig. 2 shows the elevation of the typical braced frame. The central bay uses an eccentric bracing to allow for the passage of the corridor. The adjacent bays, which lie behind the elevators in plan, are concentrically braced. An outrigger truss is located at between the 33rd and 35th Floors to engage the exterior columns.

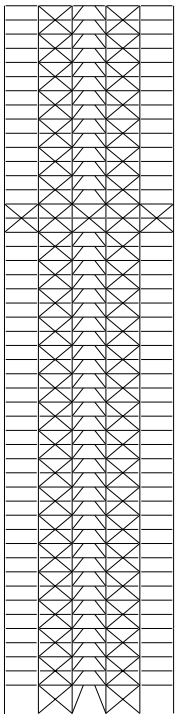

Figure 2 – Elevation of Typical Braced Frame

The building is intended to be an office structure with a live load of 80 psf. Mechanical spaces are located at the roof and outrigger floors. The curtain wall weight is 20 psf. The floor is composite construction using a 3" metal deck with 3-¼" normal-weight concrete topping. A yield stress of 50 ksi, conforming to A992 steel, was used for all structural steel members.

Wind loads were calculated based upon ASCE 7-98 (1998). The intensity of the wind loading is similar to that in New York City. The

wind loads were determined for a Basic Wind Speed of 110 mph, Exposure Category A, and Importance Factor 1.15. The gust factor was determined based upon the dynamic properties of the structure.

The first step in the study was to analyze and design this structure for the combined effects of wind and gravity loads, in much the same manner as one would design a typical project. The program ETABS, (2004) was used for both analysis and design. The steel member sizes were determined through an iterative optimization process. For the purposes of this study, serviceability criteria, such as human comfort under wind induced vibrations, were ignored. The building design of this case is referred to as Case A1.

Next, a progressive collapse upgrade to the building was performed without any considerations to the wind resiliency already present. Building A, as described above, was used. The upgrade was applied by following the procedures outlined Smith et. al. (1998). This was achieved by adding moment connections at the perimeter of the building and increasing the sizes of the perimeter columns and beams to withstand the progressive collapse induced forces. The interior column, beam and brace sizes remained as determined by the previous wind load design.

The progressive collapse design procedure assumed the loss of any one column immediately above the Ground Floor. For simplicity, the design process employed a linear-static design procedure, see Smith et. al. (1998). This was done by applying a Dynamic Load Factor of 2.0 to the gravity loads in the influence area of the lost column. The design also assumes a strength increase factor of 5% (F_y = 52.5 ksi). The analysis is performed for service level loads, 1.0 x Dead Load and 0.25 x Live Load. Members are designed elastically, with a small increase in allowable stress ratio to allow for some ductile behavior. The building design of this case is referred to as Case A2.

The third design of Building A was performed as follows: given the progressive collapse requirements for the exterior beams and columns determined in Case A2 above, redesign the interior beams, columns and braces for wind and gravity forces. For this case, the exterior member sizes remained fixed and the interior sizes were re-optimized using methods similar to those employed in Case 1. The building design of this case is referred to as Case A3.

Utilizing simple cost estimation procedures that are based on steel tonnage, the structural costs of Cases A1, A2 and A3 were evaluated. Table 1 summarizes the results of the relative costs of the structural steel framing for the three cases.

Table 1 – Wind vs. Progressive Collapse Study Results

	One-hazard-at-a-time (independent)		Multihazard
	Case A1 (Wind only)	Case A2 (Wind and Progressive Collapse)	Case A3 (Wind and Progressive Collapse)
Steel Weight (t)	8351	9564	9412
Cost ($3150/ton)	$26.31M	$30.13M	$29.65M

Table 1 shows the advantage of a multihazard design procedure. When the building was designed using a one-hazard-at-a-time approach, the total cost of the design was $30.1M. When the building was designed using a multihazard approach, the cost of design was $29.65M. This represents a savings of $480,000.00. The savings amount to almost 1.5% of the total structural steel framing cost. Note that the iterations that were needed to evaluate the multihazard design required almost no additional costs, since the designs were all done using a building model that was already available. The results of Table 1 show that the progressive collapse mitigation measures that were adopted in Case A2 included some inherent wind resiliency, which was then taken into consideration during the design of Case A3. This is the multihazard inherent resiliency that the Theory of Multihazards predicted, Ettouney et. al. (2005).

Interaction of Multihazards Systems: Wind and Seismic Example

The multihazard considerations of buildings, as presented by the Theory of Multihazards, Ettouney et. al. (2005) implies an interaction between how the building respond to hazards, i.e., the capacities to resist hazards. This means that the efficiencies of different systems that resist a given hazard depend on the magnitude and type of another hazard.

To amplify this concept, let us consider the multihazard effects of wind and seismic loads as applied to a tall building. We use a simplified version of Building A for this situation. For the simplified building, referred to as Building B, the floor-to-floor height (12'-6"), number of stories (49), and total height (625') are unchanged from Building A. The column spacing (30') and number of bays (5x5) are also unchanged, but the corner bay is no longer removed so that the building has a simple square plan. For simplification, the mechanical floors and elevator openings were removed. The plan drawing of building B is shown in Fig. 3.

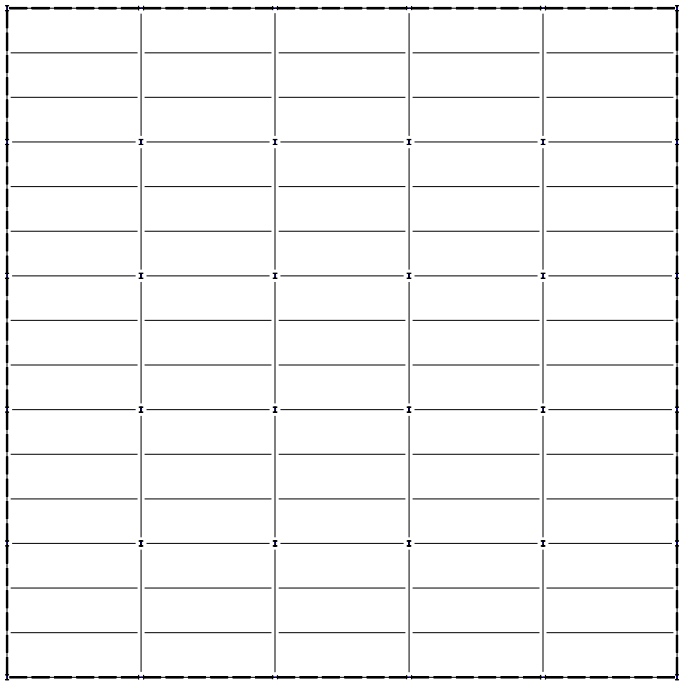

Figure 3 – Floor Plan of Building B

The lateral load system for this study is located in frames at the building exterior, rather than the interior frames used in the previous study. Two different lateral load resisting systems were examined, a Concentrically Braced Frame (CBF) and an Eccentrically Braced Frame (EBF). Typical elevations of these frames are shown in Fig. 4 and Fig. 5.

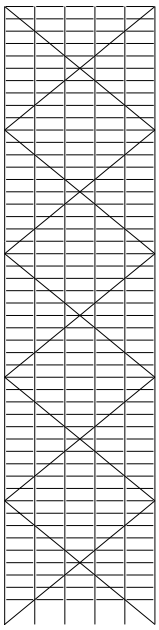

Figure 4 – Concentric Braced Frame of Building B

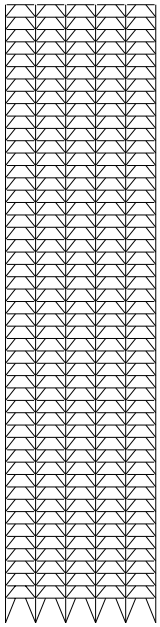

Figure 5 – Eccentric Braced Frame of Building B

The wind loads used were the same as those used for Building A (Basic Wind Speed 110 mph, Exposure Category A, and Importance Factor 1.15). Seismic loads were determined as per IBC (2000), with short and long period spectral response values of S_s = 1.0s and S_1 = 0.4s. These magnitudes of wind and seismic loading are comparable to those that would be used for a building located near the coast of North Carolina. The building is assumed to be located on soft soil, categorized as Site Class E. The building is categorized as Seismic Use Group II. Linear static analysis was used for seismic loads.

A total of four analysis/design cases, B1, B2, B3 & B4, were studied, as defined in Table 2. For the wind-only case, B1, a CBF was chosen since it is expected to be more efficient than an EBF. For the seismic only case an EBF was chosen since it is expected to be more efficient than a CBF. However, for our multihazard purpose, we are more interested in cases B2 and B4.

Note that while the site conditions were exactly the same for the 4 cases, the applied lateral base shear varied for each model. The wind gust factor was determined based upon the dynamic properties of the structure. Wind forces were reduced when the building stiffness was increased. Several factors used in the determination of the seismic loads are dependant on the structural system. The Coefficient for the Approximate Fundamental Period, C_T, has a value of 0.02 for the CBF and 0.03 for the EBF. Also the Response Modification Coefficient, R, has a value of 5 for the CBF and 8 for the EBF.

For each case, structural steel member sizes were optimized to produce the minimum weight of structural steel required to satisfy the design loads.

Table 2 – Wind vs. Seismic Test Cases and Results

Case	Frame	Applied Lateral Loads	Max Base Shear (k)	Steel Wt. (t)	Cost ($3150/ton)
Case B1	CBF	Wind Only	4137	7430	$23.40M
Case B2	CBF	Wind and Seismic	5398	7607	$23.96M
Case B3	EBF	Seismic Only	3317	7219	$22.74M
Case B4	EBF	Wind and Seismic	4818	7867	$24.78M

It is interesting that the seismic-only EBF case, B3, produced the least expensive design. Yet, when the wind is added to the design conditions, the system produced the most expensive design. For the two multihazards designs, B2 and B4, the CBF system is the least expensive. The CBF, in a multihazard condition is the preferred solution, even though it is not the preferred solution in a single hazard conditions. This leads us to this important conclusion: in a multihazards design environment, the structural systems interact in a different manner than they do in a single hazard condition (if hazards are considered separately). The savings can be substantial if this system interaction is accounted for. In the Building B case, the savings of using Case B2 instead of Case B4 is $820,000.00, or about 3.5% of the total cost.

Life-Cycle Cost Analysis: Seismic vs. Blast

One of the least considered aspects of structural design of buildings is the life-cycle cost aspects of design decisions. The life-cycle costs decisions implications also depend on whether the building is subjected to multihazards. In this example, we investigate the effects of multihazard considerations on life-cycle cost analysis. In order to simplify the example, we consider only a single time period as the basis of the example. For multi-period extensions, the discount rate must be considered, see Hawk (2003). However, since our immediate purpose is to show implications of multihazard considerations, a single period study would not lose any accuracy. Life-cycle cost analysis for the ith hazard for a single time period can be defined as

$$C_i = \sum_{j=1}^{j=N} p_{ij}\ c_{ij} \tag{1}$$

Where p_{ij} is the probability of occurrence of the ith hazard with j^{th} intensity in the time period. The number of subdivisions in the hazard intensity space is N. The cost of ith hazard with j^{th} intensity is c_{ij}. Note that, by definition, for a given hazard, i.e., constant i

$$\sum_{j=1}^{j=N} p_{ij} = 1.0 \tag{2}$$

In order to perform life-cycle cost analysis, we have to develop a method for computing the costs c_{ij} that result from different hazards.

In this example, we will study life-cycle costs of seismic and blast multihazard effects. Building B with an EBF will be used in this example. The level of seismicity was reduced slightly (S_s = 0.75s and S_1 = 0.3s). The other seismic load coefficients remained the same (C_T = 0.03, R = 8, Seismic Use Group II, Site Class E). The steel member sizes were optimized for gravity and seismic loads. After the initial optimization, the member sizes were unchanged for all subsequent analyses.

Since life-cycle cost analysis is done for a given geometry, the approach to this example is different from the previous two. Rather than optimizing member sizes for different loading and design criteria, the building design was kept constant while stress levels were investigated for various levels of loading. In this study, the building was subjected to three separate earthquake loadings (EQ1, EQ2, EQ3). These earthquakes represent 3 hazard levels (low, medium, high), with EQ2 being equivalent to the design earthquake. The spectral accelerations of the three earthquakes are listed in Table 3.

Table 3 – Seismic Hazard Spectral Accelerations

Earthquake	S_s	S_1
EQ1 (low)	0.5s	0.2s
EQ2 (medium: design level)	0.75s	0.3s
EQ3 (high)	1.0s	0.4s

Each earthquake was applied as an equivalent static load based on the response spectrum values listed in the table above. Since this study is looking at actual member stress levels, the applied loads are not reduced by the full Response Modification Coefficient, R. In order to get an estimate of the actual response level, a value of R = 1.5 is used for the investigation. Additionally, the members are checked for the load combination 1.0 DL + 0.25 LL + 1.0 EQ, rather than the usual design load combinations, which have an inherent factor of safety.

For each analysis, an accounting is made of the structural members and their stress level. Each member is categorized as undamaged, lightly damaged, moderately damaged or highly damaged depending on the stress ratio, *r*. For the definition of the stress ratio, *r*, see ETABS (2004). The relationship between stress ratio, *r*, and the damage level is shown at the left of Table 4. Note that this is a qualitative estimate; a more accurate estimate would require a nonlinear dynamic analysis, which is beyond the scope of this paper. Because beams and braces exhibit greater ductility than columns, the damage in these members is assumed to occur at higher stress levels.

A cost is assigned to each level of damage of each member. Because the failure of a column is more catastrophic than the failure of a beam or brace, the cost of column damage is greater than that of other members.

Lastly, probabilities of occurrence p_{ij} of each seismic event (EQ1, EQ2 and EQ3) during the period under consideration are estimated. All of this data is sufficient to calculate the life-cycle cost for seismic events for the building for a fixed period as in equation 1. The seismic life-cycle cost analysis is summarized in Table 4.

Next, we perform a life-cycle cost analysis for blast events. The study assumes that each blast event will affect a different envelope of the building. The low-level blast event affects only one bay, while the high-level blast event affects one entire façade. The study assumes that 50% of the members within the blast envelope sustain light damage, 30% sustain moderate damage, and 20% are highly damaged. Lastly, probabilities of occurrence are assigned to each blast event. The results of the blast life-cycle analysis are also summarized in Table 4. Note that the two life-cycle costs for seismic and blast hazards were computed separately, according to equation 1.

Next, the study investigates the effects on life-cycle cost when a blast retrofit is applied to the structure. The retrofit consists of a 24" thick concrete blast wall constructed at the perimeter of the building between the Ground and 4th Floor. The analysis model of Building B is modified to include this wall, and the complete seismic life-cycle cost analysis is performed on the retrofitted structure. The blast life-cycle cost analysis is also performed for the retrofitted structure, with the assumption that blast effects are greatly reduced. The results are summarized in Table 5.

Table 4 – Original Building B, Seismic and Blast Life-Cycle Cost

Damage	Stress		EQ1		EQ2		EQ3	
Level	Ratio	cost	No.	cost	No.	cost	No.	cost
Columns								
Undamaged	r<=1.5	$0	1764	$0	1748	$0	1700	$0
Low	1.5<r<=2	$10,000	0	$0	16	$160,000	64	$640,000
Moderate	2<r<=2.5	$50,000	0	$0	0	$0	0	$0
High	2.5<r	$150,000	0	$0	0	$0	0	$0
Beams								
Undamaged	r<=2	$0	974	$0	402	$0	176	$0
Low	2<r<=2.5	$5,000	6	$30,000	514	$2,570,000	496	$2,480,000
Moderate	2.5<r<=3	$20,000	0	$0	64	$1,280,000	308	$6,160,000
High	3<r	$50,000	0	$0	0	$0	4	$200,000
Braces								
Undamaged	r<=2	$0	1664	$0	644	$0	356	$0
Low	2<r<=2.5	$5,000	296	$1,480,000	948	$4,740,000	1056	$5,280,000
Moderate	2.5<r<=3	$20,000	0	$0	368	$7,360,000	548	$10,960,000
High	3<r	$50,000	0	$0	0	$0	276	$13,800,000
c_{ij}				$1,510,000		$16,110,000		$39,520,000
$p_{ij} \mid p_{ij} * c_{ij}$			0.8	$1,208,000	0.15	$2,416,500	0.05	$1,976,000
Seismic Cost								**$5,600,500**

Damage Level	Stress Ratio	cost	Blast1 No.	Blast1 cost	Blast2 No.	Blast2 cost	Blast3 No.	Blast3 cost
Columns								
Undamaged	r<=1.5	$0	1762	$0	1746	$0	1176	$0
Low	1.5<r<=2	$10,000	1	$10,000	9	$90,000	294	$2,940,000
Moderate	2<r<=2.5	$50,000	1	$50,000	5	$250,000	176	$8,800,000
High	2.5<r	$150,000	0	$0	4	$600,000	118	$17,700,000
Beams								
Undamaged	r<=2	$0	978	$0	964	$0	539	$0
Low	2<r<=2.5	$5,000	1	$5,000	8	$40,000	220	$1,100,000
Moderate	2.5<r<=3	$20,000	1	$20,000	5	$100,000	132	$2,640,000
High	3<r	$50,000	0	$0	3	$150,000	89	$4,450,000
Braces								
Undamaged	r<=2	$0	1954	$0	1922	$0	1176	$0
Low	2<r<=2.5	$5,000	3	$15,000	19	$95,000	392	$1,960,000
Moderate	2.5<r<=3	$20,000	2	$40,000	11	$220,000	235	$4,700,000
High	3<r	$50,000	1	$50,000	8	$400,000	157	$7,850,000
c_{ij}				$190,000		$1,945,000		$52,140,000
$p_{ij} \mid p_{ij} * c_{ij}$			0.85	$161,500	0.125	$243,125	0.025	$1,303,500
Blast Cost								**$1,708,125**
Total Cost Equation 1								**$7,308,625**

Table 5 – Blast Hardened Building B, Seismic and Blast Life-Cycle Cost

Damage Level	Stress Ratio	cost	EQ1 No.	EQ1 cost	EQ2 No.	EQ2 cost	EQ3 No.	EQ3 cost
Columns								
Undamaged	r<=1.5	$0	1760	$0	1716	$0	1680	$0
Low	1.5<r<=2	$10,000	4	$40,000	44	$440,000	84	$840,000
Moderate	2<r<=2.5	$50,000	0	$0	4	$200,000	0	$0
High	2.5<r	$150,000	0	$0	0	$0	4	$600,000
Beams								
Undamaged	r<=2	$0	850	$0	346	$0	168	$0
Low	2<r<=2.5	$5,000	130	$650,000	430	$2,150,000	492	$2,460,000
Moderate	2.5<r<=3	$20,000	0	$0	204	$4,080,000	320	$6,400,000
High	3<r	$50,000	0	$0	0	$0	96	$4,800,000
Braces								
Undamaged	r<=2	$0	1560	$0	620	$0	420	$0
Low	2<r<=2.5	$5,000	400	$2,000,000	1000	$5,000,000	844	$4,220,000
Moderate	2.5<r<=3	$20,000	0	$0	340	$6,800,000	696	$13,920,000
High	3<r	$50,000	0	$0	132	$6,600,000	384	$19,200,000
c_{ij}				$2,690,000		$25,270,000		$52,440,000
$p_{ij} \mid p_{ij} * c_{ij}$			0.8	$2,152,000	0.15	$3,790,500	0.05	$2,622,000
Seismic Cost								**$8,564,500**

Damage Level	Stress Ratio	cost	Blast1 No.	Blast1 cost	Blast2 No.	Blast2 cost	Blast3 No.	Blast3 cost
Columns								
Undamaged	r<=1.5	$0	1764	$0	1762	$0	1746	$0
Low	1.5<r<=2	$10,000	0	$0	1	$10,000	9	$90,000
Moderate	2<r<=2.5	$50,000	0	$0	1	$50,000	5	$250,000
High	2.5<r	$150,000	0	$0	0	$0	4	$600,000
Beams								
Undamaged	r<=2	$0	980	$0	978	$0	964	$0
Low	2<r<=2.5	$5,000	0	$0	1	$5,000	8	$40,000
Moderate	2.5<r<=3	$20,000	0	$0	1	$20,000	5	$100,000
High	3<r	$50,000	0	$0	0	$0	3	$150,000
Braces								
Undamaged	r<=2	$0	1960	$0	1954	$0	1922	$0
Low	2<r<=2.5	$5,000	0	$0	3	$15,000	19	$95,000
Moderate	2.5<r<=3	$20,000	0	$0	2	$40,000	11	$220,000
High	3<r	$50,000	0	$0	1	$50,000	8	$400,000
c_{ij}				$0		$190,000		$1,945,000
$p_{ij} \mid p_{ij} * c_{ij}$			0.85	$0	0.125	$23,750	0.025	$48,625
Blast Cost								**$72,375**
Total Cost Equation 1								**$8,636,875**

Comparing the total life-cycle cost of the original building and the blast hardened building shows that the blast hardened building is $1,328,250 more expensive than the original building, nearly an 18% increase in cost. This means that the hardening of the building actually costs more than if the building is left in its original state. This result is due to the fact that by hardening the lower few floors of the building to resist blast effects, the building seismic properties are changed. The changes include lowering the natural period of the building, thus increasing the seismic demand on the building. These changes increased seismic vulnerability of the building, thus the damage levels of different building components are changed. Note that the blast costs in the hardened building decrease considerably; however, the increase in seismic costs more than offset the decreased blast costs. The blast hardening of the lower floors is in conflict with the seismic need for lower period, which increases the life-cycle cost of the building; for more discussion, see Ettouney, et. al. (1998) and Ettouney (2001). This shows that multihazard life-cycle cost considerations can result in increased costs if the two hazards have conflicting demands, as in this case. Ettouney, et. al. (2005) have predicted this phenomenon. They also predicted that as the two hazards have consistent demands, the life-cycle costs will decrease for the multihazard situation.

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

Multihazards considerations for tall buildings were considered in this paper. The inherent capacities of tall buildings to resist multihazards were evaluated for a wind and progressive collapse example. Next, a wind and seismic example showed that the efficiency of structural systems can differ, depending on the number and level of hazards under consideration. Finally, a life-cycle cost example showed that those life-cycle costs can increase immensely if the different hazards and the underlying building systems interact in a conflicting manner.

It is concluded that a considerable cost implication can result from considering, or ignoring, multihazards effects. In the examples shown above, those cost implications ranged from 1.5% to 18%. In tall buildings, this would be a major cost that can be easily saved, if proper multihazard considerations are followed during the design stage of the building.

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