CSCE 2011 General Conference - *Congrès générale 2011 de la SCGC*

Ottawa, Ontario

June 14-17, 2011 / 14 au 17 juin 2011

Decision Making Tool for Seismic Retrofit of Non-ductile Reinforced Concrete Building

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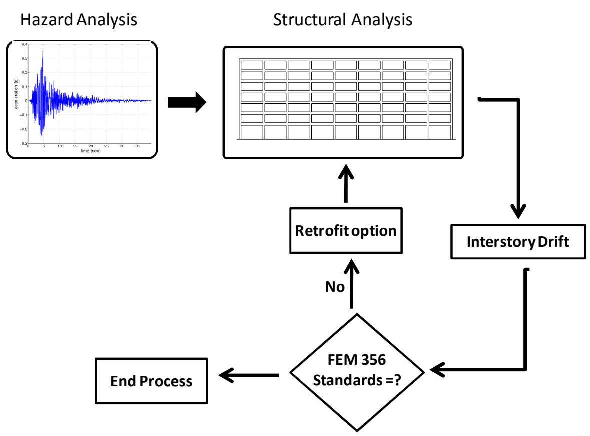
Abstract: Increasing seismicity and reported damage highlights a need for seismic vulnerability assessment of older (non-code conforming) buildings and retrofitting to reduce risk to public safety. The vulnerability can be reduced by developing a seismic retrofit program. This paper analytically investigates the effectiveness of different retrofit alternatives to upgrade performance level of existing non-ductile reinforced concrete (RC) building. A seven-story reinforced concrete building located in western U.S. and designed according to 1964 Los Angeles City Building Code is selected as a case study. The seismic performance of the structure is examined for moderate and high hazard levels, namely: 10%, and 2% probability of occurrence in 50 year period. Two rehabilitation strategies, fiber reinforced polymer, and steel braces, are studied. The enhanced seismic performance of the building is evaluated using both nonlinear static and nonlinear dynamic analysis. Desired retrofitting technique is selected employing decision making tool.

1. INTRODUCTION

Recent earthquakes such as Saguenay in Canada (1988); Loma Prieta in U.S (1989); Northridge in U.S (1994); Kobe in Japan (1995), and Golcuk-Izmit in Turkey (1999) demonstrated vulnerability of existing reinforced concrete buildings designed according to pre1970 codes. This is due to the fact that the design philosophy of codes before 1970s focused on strength-based approach without enforcing proper reinforcement detailing of members and overall ductility of the structure. The wide spacing of transverse reinforcement, discontinuity of positive reinforcement in beam and slab, and short lap-splice are seismic deficiencies commonly suffered by structures designed to the then available codes (Ghobarah 2000). This implies limited energy dissipation capacity during seismic event, suggesting potential seismic threat is constituted by such structures to public safety. Motivated by damage and failure observations during the events, modification to seismic design provisions occurred that lies on making ductile detailing compulsory for reinforced concrete (RC) structures besides to lateral load resistance requirements. However, as major stock of aging facilities worldwide were constructed before considerable advancement to design codes is instituted, there is a need for risk management strategies to accommodate susceptibility of older designed RC structures to earthquake hazards (Tesfamariam *et al*. 2010). In that trend, rehabilitation of seismically deficient structures plays an effective role to upgrade system level performance of vulnerable structures to match modern standards. In fact, several of the major earthquakes illustrated the improved seismic performance of retrofitted structures, and addressed the importance of reducing potential seismic risk through retrofitting (Bai *et al*. 2007).

This study investigates effectiveness of adapting retrofit options to improve seismic performance of typical 1960s RC structure. The vulnerability evaluation of the representative concrete structures of older designed building is carried according to the *Prestandard and Commentary for the Seismic Rehabilitation* *of Buildings* (FEMA 356). This engineering approach outlines criteria for performance-based evaluation of existing structures and guidelines for seismic design of retrofit options (Figure 1). The design objectives are expressed in term of achieving desired performance targets for various earthquake return periods. The performance targets are classified based on drift limits to control the damage sustained by structure during seismic event. The target performance levels of FEMA 356 include Immediate Occupancy (IO), Life-Safety (LS), and Collapse Prevention (CP). IO performance level requires that the structure sustains minor amount of damage. LS performance level is defined as that structure sustained significant amount of damages, but retained margin against collapse. Structure at CP provides resistance to gravity loads with little margin against collapse. The three hazard levels are related to maximum interstory drift, as a damage indicator parameter, of 1%, 2%, and 4%, respectively. In FEMA 356, *Basic Safety Objective (BSO)* requires that LS and CP performance levels are achieved for hazard level of 10% and 2%, respectively.



		Operational	Immediate Occupancy	Life Safety	Collapse Prevention	Probability of Exceedance
Drift limits		(0.6%)	(1.0%)	(2.0%)	(4.0%)	
Earthquake Return Period	72	a	b	c	d	50%/50 years
	225	e	f	g	h	20%/50 years
	474	i	j	k	l	10%/ 50 years
	2475	m	n	o	p	2%/ 50 years

Notes:
1. Each cell in the above matrix represents a discrete Rehabilitation Objective.
2. The Rehabilitation Objectives in the matrix above may be used to represent the three specific Rehabilitation Objectives defined in Sections 1.4.1, 1.4.2, and 1.4.3, as follows:
k + p = Basic Safety Objective (BSO)
k + p + any of a, e, i, b, f, j, or n = Enhanced Objectives
o alone or n alone or m alone = Enhanced Objective
k alone or p alone = Limited Objectives
c, g, d, h, l = Limited Objectives

(a) (b)

Figure 1. a) Overview of the study methodology b) performance level
(reproduced from FEMA 356 2000)

The outcomes of this study shall provide insight to the seismic performance of RC building designed by the standards of 1960s, and assess the effectiveness of alternate retrofit options to accommodate susceptibility of existing non-ductile RC structure. Two rehabilitation patterns are considered, namely: Steel bracing, and FRP wrapping. Nonlinear time history analysis and static pushover analysis were conducted to assess the enhanced performance of rehabilitated structure. A decision making tool for retrofit selection is also illustrated.

2. DESCRIPTION OF CASE STUDY REINFORCED CONCRETE BUILDING

Seven-story hotel building located at Van Nuys city of Los Angeles county, California, is selected as study case structure (Cornell *et al.* 2005). The hotel building was designed during 1965 according to the 1964 Los Angeles City Building Code, and constructed in 1966. The building plan dimensions are 19 m (62 ft, 8 inch) by 46 m (151 ft, 2 inch) in the north-south and east-west directions, respectively. The building is 19.8 m (65 ft) tall with uniform mass and stiffness distribution, and the structural system is regularities in plan and elevation. The structural configuration of the building consists of interior flat-slab system and perimeter moment resisting frame with non-ductile detailing. The building foundation system consists of pile cape supported by two to four groups of concrete friction piles. The columns are founded on the centerline of the pile cap. The caps are connected together using tie beams and grade beams. Each pile is of 600 mm (24 in) diameter and 13 m (40 ft) depth. Each pile was designed to provide vertical capacity of over 445 kN (100 kips) and lateral capacity of 89 kN (20 kips). The site geology consists of fine sand silts and silty fine sand. The concrete compressive strength of columns are 35 MPa (5 ksi) at the first story level, 28 MPa (4 ksi) at the second story level, and 20 MPa (3 ksi) at the third to the seventh floor. Beam and slab concrete compressive strength is 28 MPa (4 ksi) at the second floor, and 20 MPa (3 ksi) for the third to the roof. The reinforcement bars of columns are of grade 60. The beam and slab reinforcement bars are of grade 40.

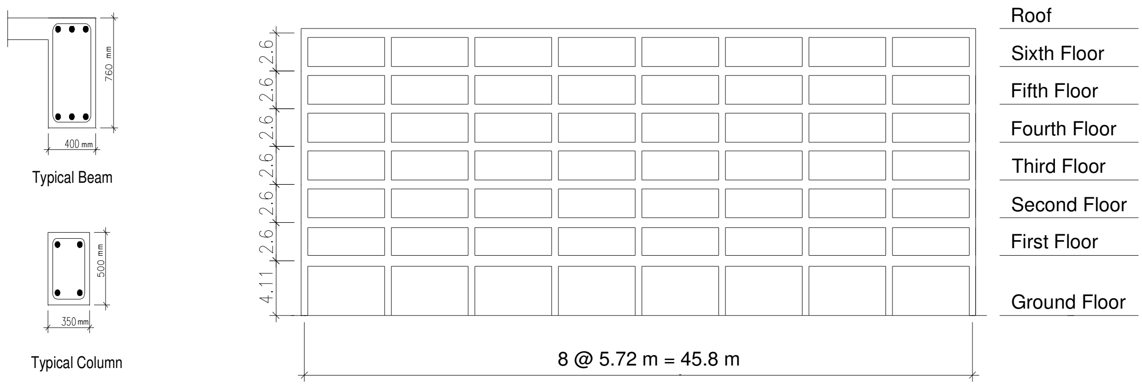
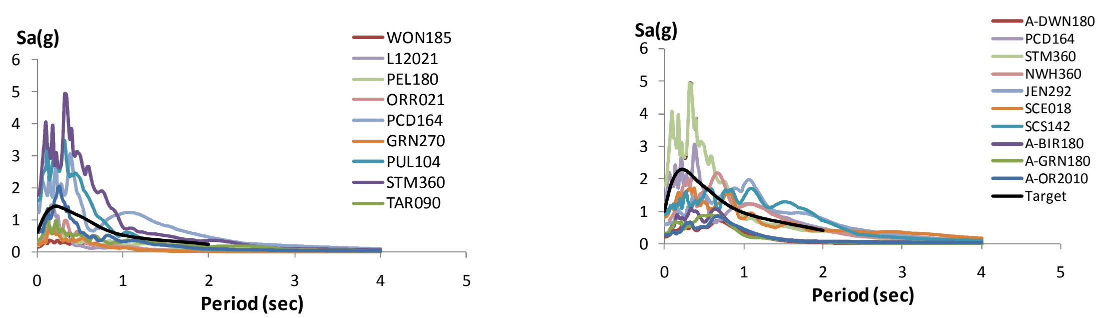


Figure 2. Elevation view of the case study frame

3. SELECTION AND SCALING OF GROUND MOTION RECORDS

The seismic performance of the RC building is evaluated through series of nonlinear time history analysis. This requires that the selected ground motion records should reflect the expected magnitude, fault distance, and soil condition at the site of the structure (Cornell *et al.* 2005). The quantification of the parameters to identify proper ground motion record is obtained through de-aggregating the site hazard spectrum. In this study, the uniform hazard spectrums at the site were generated using USGS software [available at: http://earthquake.usgs.gov/hazards/designmaps/javacalc.php] considering 2% and 10% probability of exceedance in 50 year return period. Detailed information on the de-aggregation process of the hazard spectrums and the identification of proper ground motion records to accompany intensity measure is provided in Cornell *et al.* (2005). In the current study, a set of ten records is used to represent each hazard level. The ground motion records to accompany 10% and 2% hazard levels and corresponding target UHS are shown in Figure 3. Each set of time history records is scaled to match the intensity measure of the considered hazard level at the fundamental first period (T_1) of the building. The intensity measure used in this study is the spectral acceleration (S_a) at specified damping level of 5%. The scaling process of the ground motion records reflects the inputted seismic energy at the assumed hazard level, and eliminates the variability in the records (Liel 2008).



(a)10% hazard level (b) 2% hazard level

Figure 3. Shows selected ground motions to represent hazard curves

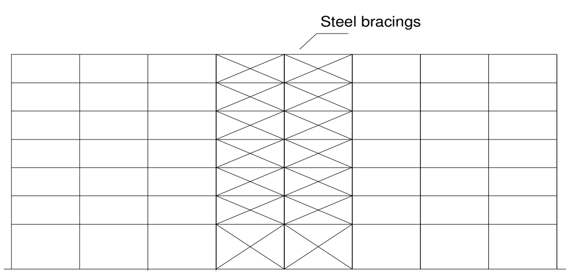
4. MODELLING OF THE CASE STUDY REINFORCED CONCRETE BUILDING

Simulation of the structural system to assess its seismic performance requires the development of mathematical models that captures the nonlinear force-deformation properties of the building. In this study, a two dimensional analytical model of the exterior frame is modelled in Seismostruct (2010). Seismostruct is a finite-element analysis program, capable of performing nonlinear time history analysis, pushover analysis, and eigenvalue analysis. The program uses fiber-element approach to capture the spread of plasticity along the section depth and member length. In this approach, the effect of material nonlinearity is taken into account by dividing the member cross-section into fibers to which the appropriate material constitutive formulations are assigned.

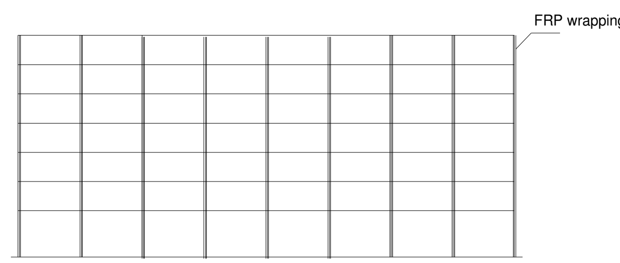
Computational model of the exterior frame (as shown in Figure 2) is created to investigate the seismic performance of the building. The model is considered adequate to simulate the dynamic characteristics of the building because the components of the exterior frame are the primary elements responsible for the lateral load resistance of the structure. This is also to take advantage of the regular and symmetrical building configuration. The frame components were modelled using forced-based inelastic frame elements. Concrete materials are modelled using uniaxial nonlinear constant confinement model, as introduced by the material library in Seismostruct. Steel materials are modeled using uniaxial bilinear stress-strain model with constant elastic range for various loading stages. The modulus of elasticity and strain hardening parameter of the steel were considered as 200,000 MPa and 0.004, respectively. The confinement effect of FRP-wrapping was simulated using uniaxial nonlinear variable confinement model for RC with FRP. The floor mass was considered the equivalent mass to the distributed loads based on the tributary area at each story. The masses were lumped to the column joints.

5. SEISMIC PERFORMANCE EVALUATION

Seven-storey existing frame is studied to evaluate the seismic vulnerability constituted in typical RC buildings designed according to pre1970 codes. The drift limits to communicate the damage state of the structure for a stated hazard level were considered as that provided by FEMA 356. Two rehabilitation strategies were selected to upgrade the structural characteristics of the existing frame. The first retrofit scheme is to utilize steel bracings to strengthen the overall lateral load supply of the structure, and reduces the demand on existing structural components. The second retrofit method is to wrap column members with FRP sheets to treat design and detailing deficiencies. This is to enhance the deformation capacity of the retrofitted members so it exhibit pre-defined ductility level without reaching ultimate strain.



(a) Steel bracing



(b) FRP

Figure 4. Shows the locations of rehabilitated schemes

5.1 Retrofit option 1 (Steel bracing)

The addition of steel braces is a common retrofit scheme to increase the stiffness and control lateral drift of the structure. As steel is characterized by its high strength to weight ratio, it is considered as an effective mean to enhance the lateral load capacity of the structure without altering its mass. This is in addition to that steel braces are easy to install, accommodate frame openings, and their application involves minimum disturbance to the occupant in case they are applied to external frames. Because hollow sections are featured with their effective slenderness ratio and high compressive capacity, square tube steel sections with width of 114 mm and thickness of 8 mm were selected as bracing members. The tensile strength is considered as 350 MPa, and the slenderness ratio is 115.

5.2 Retrofit option 2 (FRP wrapping)

This technique is frequently used as a mean of ductility enhancement to seek the increment in plastic behaviour of individual components and, ultimately, the structure. Adapting FRP intervention method allows structural components to exhibit high chord rotation suggesting increased reliability of the structure against earthquake actions. It is a useful approach to upgrade the limited flexural deformability of structural elements due to inadequate concrete confinement. In the present study, the columns are wrapped with two sheets of CFRP. The thickness of each layer is selected as 0.165 mm. The initial elastic modulus and ultimate strain of the FRP jacket is 230000 MPa and 2%, respectively.

6. **PUSHOVER ANALYSIS**

Pushover analysis is conducted to examine the lateral load capacity and deformation pattern of unretrofitted and retrofitted frame structure. The inverted triangular load pattern was considered as a distribution of the base shear forces over the height of the structure. The analysis is performed with displacement control limit up to failure. The objective of the analysis is to generate relationship between global response parameter (i.e. drift at roof level) to base shear forces. This demonstrates the contribution of alternate intervention scheme on the overall performance of the structure. Comparison of pushover curves of rehabilitated and existing structure is shown in Figure 5. The response analysis shows that each retrofit scheme affects the global performance characteristics of the structure to varying degrees.

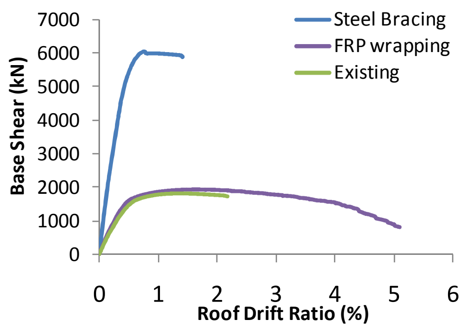


Figure 5. Pushover curves for retrofitted and unretrofitted structure

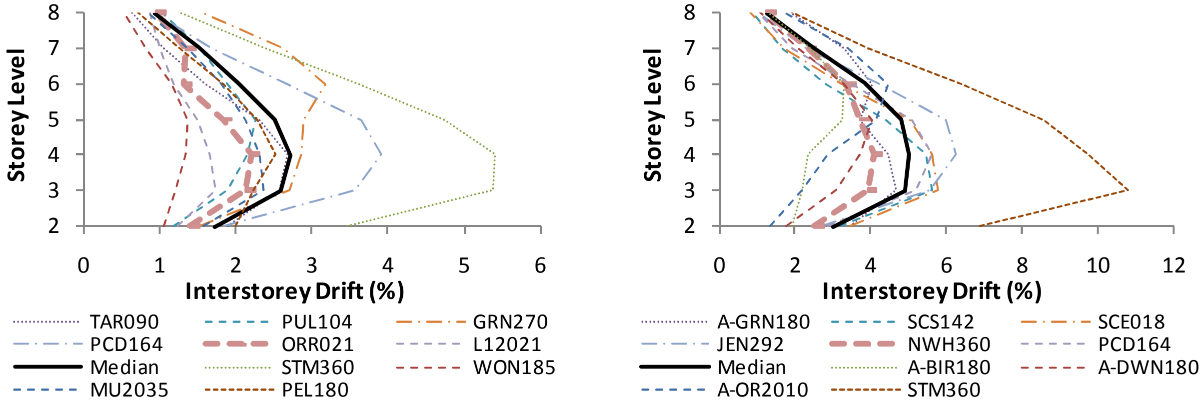
The introduction of steel braces enhanced the stiffness characteristics of the structure. This was associated with increase in the base shear capacity up to 6000 kN, as compared to 1800 kN for unretrofitted structure. This aspect indicated that most of the lateral resistance of the structure was provided by the steel bracing. Adopting FRP-wrapping did not significantly contribute to the strength and stiffness of the original structure. The increased in base shear capacity is estimated to be of 8.3%. However, incorporating this intervention method resulted in slower strength degradation and improved ductility of the structure.

7. **NONLINEAR TIME HISTORY ANALYSIS**

Nonlinear time history analyses were performed to quantify the dynamic response parameters of the retrofitted and unretrofitted study case building. The analyses were carried through incorporating ground motion records to the time domain of structural models to compute nonlinear force-deformation properties of the structure at each time increment. This analysis is a useful tool to investigate the inelastic response of structural components as it accounts for inelastic actions (e.g. flexural yielding) and the corresponding changes in the strength and stiffness of the structure. This implies capability of this analytical approach to capture the changes in dynamic properties of the structure during inelasticity.

7.1 Unretrofitted building

Seismic demand of the existing frame is quantified using inter-storey drift as performance indicator. This performance indicator accounts for flexural demand, or amount of rotation, placed on the columns (Ghobarah 2000). The inter-storey drift is defined as the relative displacement with adjacent storey divided by the storey height. Figure 6 provides the median inter-story corresponding to the considered hazard levels. It is indicated that the maximum median inter-storey drift experienced by the structure exceeds the limits of LS and CP performance level. This indicates the need of retrofitting the structure to meet BSO as per global-level evaluation of FEMA 356.

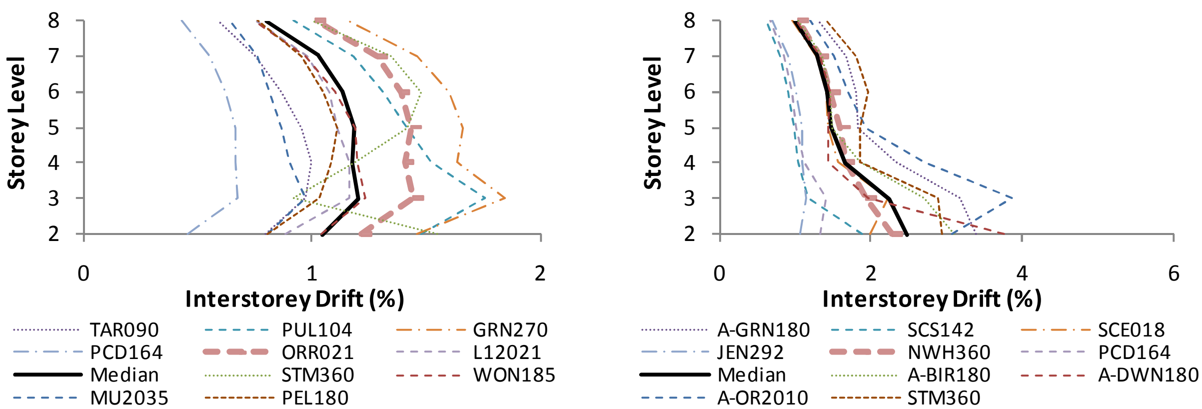


(a) 10% hazard level (b) 2% hazard level

Figure 6. Shows inter-storey drift for unretrofitted frame

7.2 Retrofitting using steel bracing

Because of the confirmed deficiencies of the existing frame, steel X-bracings were introduced to explore their effectiveness of controlling the seismic demands of the structure. Figure 7 shows the improved performance attained through utilizing steel braces to protect the system against earthquake hazards. It is indicated that introducing steel braces resulted in improved performance of the structure. Both the drift limits of LS and CP performance levels are maintained, indicating suitability of the retrofit scheme to satisfy BSO as recommended by the global-level criteria of FEMA 356.

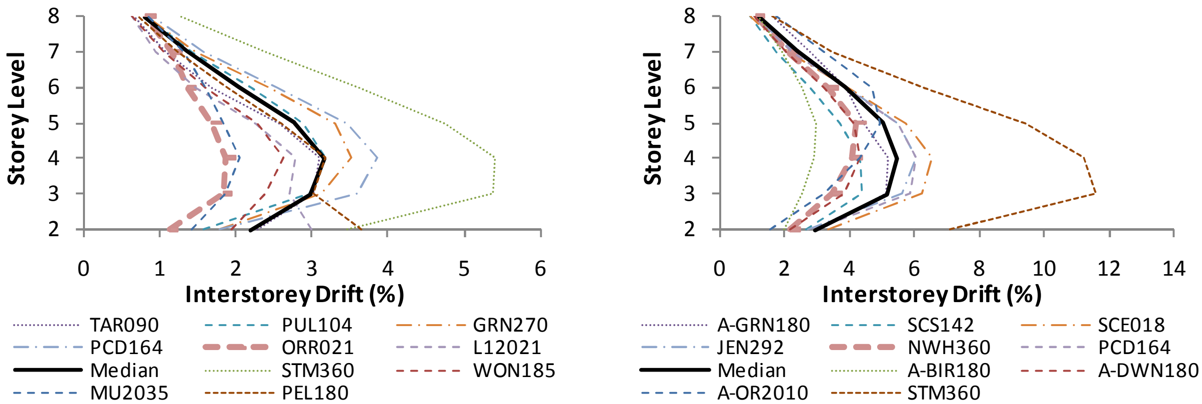


(a) 10% hazard level (b) 2% hazard level

Figure 7. Shows inter-storey drift for retrofitted frame with steel bracing

7.2 Retrofitting using FRP

The objective of this intervention method is to improve the overall performance of the structure through upgrading the limited ductility of column members. Figure 8 indicates the attained response through adopting FRP-wrapping. The results indicate that introducing this retrofit scheme did not alter the drift profile of the structure. In fact, the technique contributed to more structural demand being placed on the first three stories as compared to the unretrofitted frame. This concludes that this intervention method did not satisfy the BSO as suggested by FEMA 356.

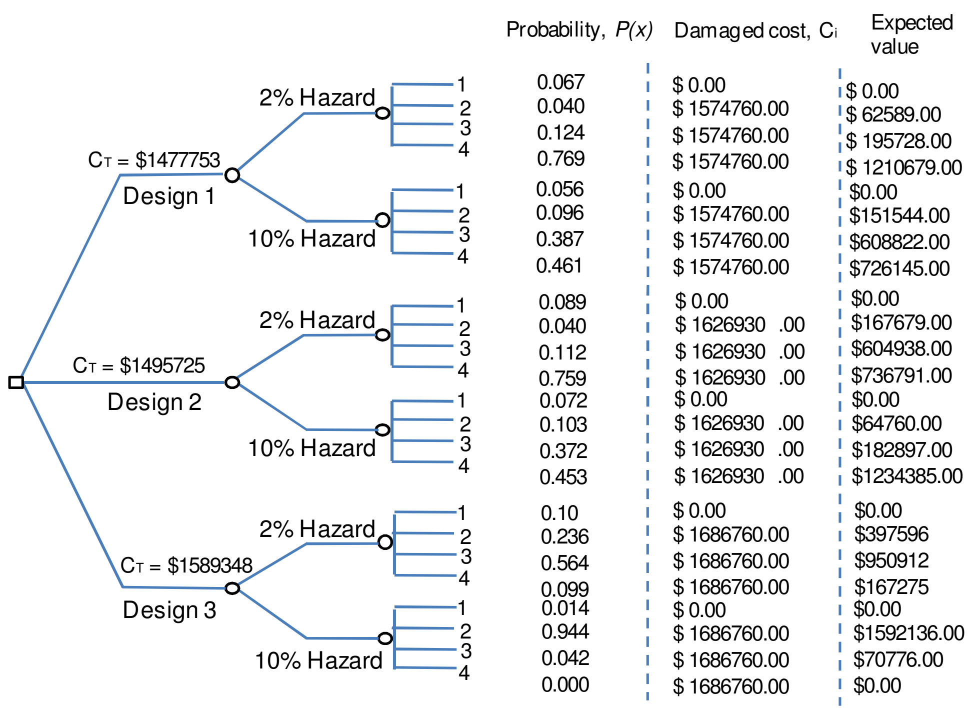


(a) 10% hazard level (b) 2% hazard level

Figure 8. Shows inter-storey drift for retrofitted frame with FRP wrapping

8. RETROFIT SELECTION DECISION MAKING TOOL

Decision tree is a decision support tool that uses a tree-like graph or model of decisions and their possible consequences, including chance event outcomes, resource costs, and utility (JCSS 2008). Decision trees can be used to evaluate best retrofitting option (Matin 2006). A proposed decision tree for retrofit selection is shown in Figure 9. In Figure 9, squares indicate decision nodes (in this case to retrofit or not retrofit), and if retrofit, which retrofit technique to select. Number of branches generated from the decision nodes depends upon the number of actions considered by the decision maker. Small circles identify chance nodes. They represent an event that can result in two or more uncertain outcomes.



1 = No damage
2= Damage exceeding IO
3= Damage exceeding LS
4= Damage exceeding CP

Design 1= Unretrofitted Structure
Design 2= Structure with FRP
Design 3= Structure with Steel bracing

IO=Immediate occupancy
LS= Life safety
CP= Collapse prevention

Figure 9. Decision Tree used for retrofit selection

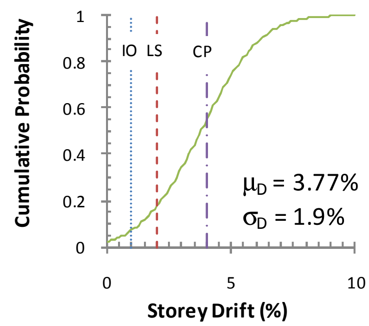
The decision tree proceeds from left to right; the later events/ decisions following the earlier events/decisions. Each branch of the decision node leads to an event fork. There is uncertainty associated with each event. At the very end of each branch, there will be corresponding damage cost and probability of occurrence. The events considered at each chance node must be mutually exclusive and collectively exhaustive, i.e. the sum of their probabilities must be equal to one (JCSS 2008). Probabilities ($p(x)$) of each event can be calculated by developing normal distribution curves. If X is the event which is assumed to be a continuous random variable, and $f(x)$ is the probability density function, then the probability of occurrence of X in interval of a and b is given by,

$$P(a < x \leq b) = \int_a^b f(x)dx \quad [1]$$

The mean (μ_D) and standard deviation (σ_D) of the maximum drifts values computed from Figures 6-8 are summarized in Table 1. For each hazard levels (2% and 10%), the corresponding probability of exceeding no damage (ND), collapse prevention (CP), life safety (LS), immediate occupancy (IO) drift levels can be computed as shown in the cumulative distribution curves Table 1.

Table 1: Mean and standard deviation of the drift demand of different hazard levels and retrofitting options

Retrofitting type	Hazard level	Mean drift (μ_D)	Standard deviation (σ_D)	R^2
Unretrofitted	10%	3.83	1.78	0.95
Unretrofitted	2%	6.89	3.94	0.97
Steel Bracing	10%	1.56	0.25	0.89
Steel Bracing	2%	2.49	1.17	0.89
FRP Wrapping	10%	3.77	1.90	0.94
FRP Wrapping	2%	7.27	4.66	0.97



The expected value (EV) associated with each decision action can be computed considering the probabilities $p(x)$ and cost associated with that decision action (C):

$$EV = \sum_{i=n}^{n} p(x)_{,i} C_i \quad [2]$$

(Tesfamariam and Sanchez-Silva 2010)

Finally the total expected cost for each hazard level is multiplied by the probability of occurrence of earthquake of each hazard level (10% and 2%) and summed up to get the total expected damage cost. For cost estimation, the cost for including steel bracing in each bay is assumed to be $8,000 and cost of FRP per m^2 is $173. The total cost of installing Steel bracing becomes $112,000 and FRP wrapping is $52,170. The cost of existing building per square feet is $ 23.86 (FEMA 156). The total area of the building is 6200 sq. meter. All the calculated values are included in decision tree. After making the decision tree it's easy for the engineers to make decisions depending on the selection criterion. Considering the minimum expected cost as selection criterion, the steel bracing retrofitting shows minimum cost of $1,495,725. Structure with FRP wrapping shows total cost larger than the Structure with steel bracing.

9. DISCUSSION AND CONCLUSION

The contribution of retrofit options to upgrade performance level of existing non-ductile RC building designed according to pre1970 standards was evaluated. The frame was rehabilitated using two intervention methods: steel X-bracing and FRP wrapping. The performance of the retrofitted and unretrofitted structure was investigated using both nonlinear static and time history analysis. The global evaluation criteria of FEMA 356 were employed to assess the upgraded seismic performance of the retrofitted frames.

Introducing concentric steel braces resulted in significant decrease in the seismic response of the building. This reduction indicates suitability of the retrofit scheme to act as additional lateral load resisting system, and upgrade the stiffness and strength of the building. This aspect suggests less rotation demand is placed on the columns of braced frame implying reliability of the structure against stability (or nonlinearity) problems related to P-Δ effect. It was also demonstrated in the study that utilizing steel bracing is efficient to control drift demand, and upgrade performance level of seismically deficient structure to meet modern standards. However, the increase in the frame lateral load capacity was associated with reduction in the deformation capacity of the structure, indicating brittle mode failure of the structure. Furthermore, different inter-storey drift profile is observed in retrofitted frame with steel bracings as compared to unretrofitted frame. This can be attributed to the fact that the deformation mechanism of moment resisting frame is characterized by beam shear behaviour, whereas the deformation of braced systems is characterized by flexure behaviour of beam.

The study also investigated the effectiveness of treating non-ductile columns using FRP wrapping. The results indicated considerable enhancement in the overall energy dissipation capacity of the structure. This indicates suitability of the intervention method to accommodate detailing deficiencies and upgrade flexural deformability of columns. This suggests suitability of the retrofit scheme to alter failure pattern of the structure. However, adopted FRP-wrapping did not reduce the seismic demand of the structure. This is due to the fact that this technique intends to upgrade plastic behaviour of structure without altering the stiffness.

10. **REFERENCES**

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