A LIFE-CYCLE COST ASSESSMENT OF SEISMIC DESIGN STRATEGIES FOR STEEL MOMENT FRAMES WITH EMPHASIS ON FAILURE MODE CONTROL

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1. INTRODUCTION

The objective of this study is to conduct a life-cycle cost analysis (LCCA) of steel moment frames (SMF) designed with a focus on failure mode control. To implement failure mode control, the theory of plastic mechanism control (TPMC) is employed. TPMC is an energy-based method that utilizes the upper bound plastic theorem and balances the energy associated with the internal and external works done by the structure during the development of a plastic mechanism in a seismic event [1]. This method ensures the achievement of a global mechanism, wherein all plastic hinges form in all beams and at the base of the columns in the first story of a moment frame.

2. METHODOLOGY

The expected total life-cycle cost (TLCC) for seismic hazard represents the average cumulative net present value of a building, encompassing both initial costs and anticipated expenses associated with damage over the expected lifespan [2].

$$C(t,X)=C_0(X)+C_{LC}(t,X) \tag{1}$$

Where C_0 is the initial cost, C_{LC} is the expected life-cycle cost of the structure, and X is the design variable vector. The C_{LC} is calculated for the lifetime t of the structure as follows [3]:

$$C_{LC}(t,X)=\frac{\nu}{\lambda}\left(1-e^{-\lambda t}\right)\sum_{i=1}^{n} C_{LC.i}P_i(t,X) \tag{2}$$

Where $C_{LC.i}$ is the expected damage cost for the ith limit state (LS), n is the number of LSs, λ is the constant discount rate per year, ν is the annual seismic hazard occurrence rate assuming a simple Poisson process distribution, and P_i is the failure probability of the ith damage state as follows [4]:

$$P_i(t,X)=\frac{-1}{\nu t}\left[\ln\left(1-P_t(d>DI_i)\right)-\ln\left(1-P_t(d> \right.\right. \tag{3}$$

Where $P_t(d>DI_i)$ is the t-year failure probability of exceeding of ith damage state (DS), quantified as:

$$P_t(d>DI_i)=1-\left(1-P_{Afi}\right)^t \tag{4}$$

Where P_{Afi} is the annual probability of exceedance, which can be calculated using seismic fragility function, $P[d>DI_i \vee \Im=a]$, and seismic hazard, $H(a)$ as follows [5]:

$$P_{Afi}=\int_0^{\infty} P[d>DI_i \vee \Im=a]\left|\frac{dH(a)}{da}\right|da \tag{5}$$

The seismic fragility function can be described as cumulative lognormal distribution as follows:

$$P[d>DI \vee \Im=a]=\Phi\left[\left(\ln a-\ln \hat{S}_a\right)/\beta_T\right] \tag{6}$$

Where Φ is the standard normal cumulative distribution, $\hat{S}_a$ is the median value of the fragility of the structure, and β_T is the standard deviation of the natural logarithm of the mean DI demand of the structure in ith DS under the earthquake with a spectral acceleration a. The dispersion parameter βT reflecting total uncertainties for each LS is estimated as follows [6]:

$$\beta_T=\sqrt{\beta_{RTR}^2+\beta_{DR}^2+\beta_{TD}^2+\beta_{MDL}^2} \tag{7}$$

Where β_{RTR} is the record-to-record uncertainty, and β_{DR}, β_{TD}, and β_{MDL}, with values of 0.1, 0.2, and 0.2 respectively, represent the design requirements-related, test data-related, and modeling uncertainties. The seismic hazard, $H(a)=P[\Im>a]$, is the annual probability of exceeding specified levels of IM, a, as follows:

$$H(a)=k_0 a^{-k} \tag{8}$$

Where k_0 and k are the constants depending on the site of the structure. Figure 1 illustrates the LCCA flowchart employed in this research. It is noted that in Eq.(2), $C_{LC.i}$ itself involves some other damage cost components as follows [7]:

$$C_{LC.i} = C_S^i + C_{NS}^i + C_R^i + C_C^i + C_I^i + C_F^i \quad (9)$$

Where for the ith LS the components of $C_{LS.i}$ described and quantified as Table 1.

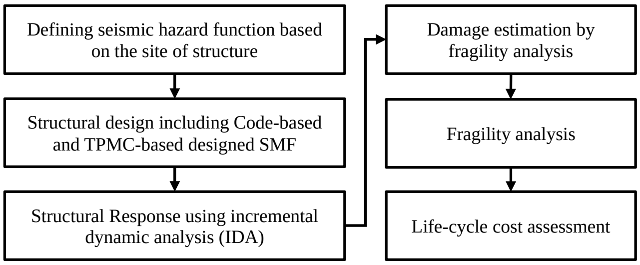

Figure 1. Methodology of life-cycle cost assessment used in this study.

Table 1. Costs related to ith limit state [7, 8].

Cost	Description	Basic cost (B_j ¿	Relation
C_S^i	Structural damage	\$1500/m^2	$B_S \overline{DI}_i A_f$
C_{NS}^i	Non-structural damage	\$500/m^2	$B_{NS} \overline{DI}_i A_f$
C_R^i	Cost of rent loss	\$10/month/m^2	$B_R A_r T_{Ri}$
C_C^i	Cost of commercial loss e.g. downtime of working	\$2000/m^2	$B_C A_r T_{Di}$
C_{MI}^i	Cost of minor injuries	\$2x10^3/person	$\rho B_{MI} \overline{DI}_i$ I
C_{SI}^i	Cost of serious injuries	\$2x10^4/person	$\rho B_{SI} \overline{DI}_i R$
C_F^i	Cost of human fatalities	\$2.8x10^6/person	$\rho B_F \overline{DI}_i R$

$\overline{DI}_i$ = mean damage state (see Table 2)

A_f = Floor area

A_r = Rentable floor area

T_{Di}= Loss of time in ith LS (see Table 2)

Occupancy rate $\rho = 0.02$

$R_{MI.i}$, $R_{SI.i}$ & $R_{F.i}$= minor injury, serious injury and human fatality rate in the ith LS (see Table 2)

3. PRACTICAL EXAMPLES

The SMFs are designed according to both current seismic provision codes and by TPMC. These designed structures are displayed in Figure 2. To compare the seismic behavior of these structures and validate the TPMC design procedure, pushover analyses were conducted, with the results presented in Figure 3.

4. RESULTS

The seismic fragility of both the SMFs designed according to current seismic provision codes and those designed by TPMC was assessed through incremental dynamic analyses (IDA). Figure 4 displays the scaled spectra of the considered earthquake records and the ASCE 7-22 spectrum. The IDA results are presented in Figure 5. By defining limit states as presented in Table 2, a fragility analysis was conducted, and the fragility curves are illustrated in Figure 6. The dispersion parameters and median of the failure spectral accelerations are reported in Table 3. Using the hazard function shown in Figure 7, a LCCA was performed.

The expected economic losses throughout the life cycle, were compared between the two SMFs as shown in Table 4 and Figure 8. The results demonstrate that the SMF designed using TPMC, despite having higher initial costs, ultimately reduces the expected costs of the structure due to its superior seismic performance.

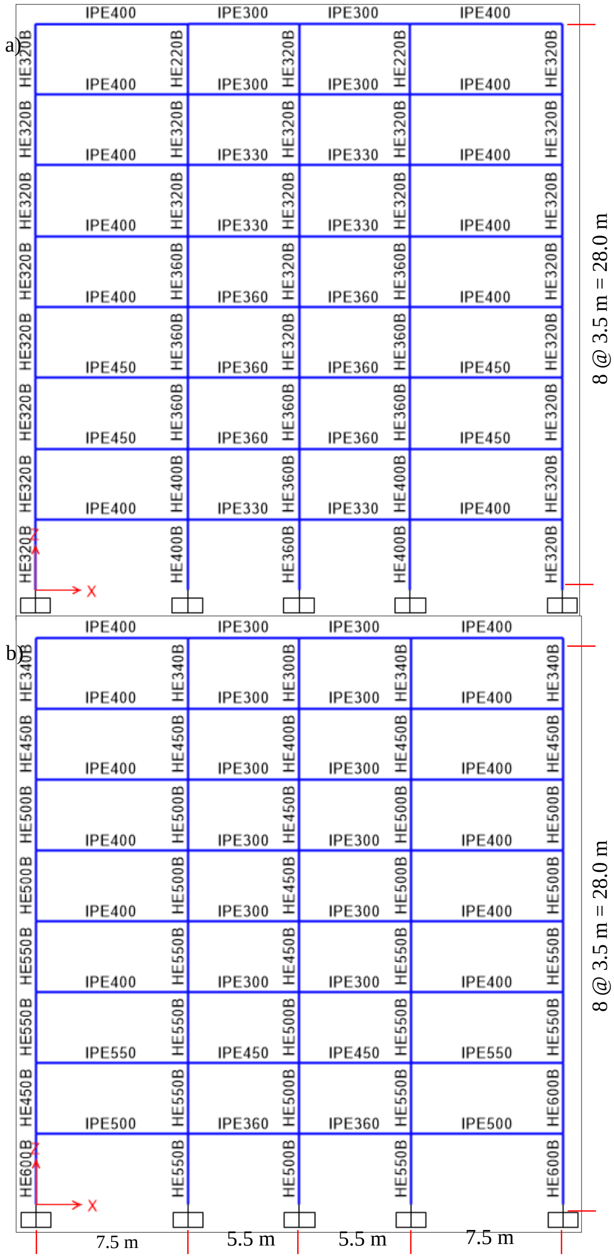

Figure 2. SMFs are designed using steel $F_y = 310.66\ MPa$, seismic parameters of $I_e = 1$, $R = 7.0$, $C_d = 5.5$, and site of structures $S_{DS} = 1.0$, $S_{D1} = 0.6$ for a) Code-based design, and b) TPMC-based design with design drift ratio, $\theta = 0.04$.

Table 2. Definition of limit state in term of inter-story drift ratio (%) [9]

Damage state (DS)	DS in term of DR (%)	$\overline{DI}_i$ (%)	T_{Di} (%)	T_{Ri} (%)	$R_{MI.i}$	$R_{SI.i}$	$R_{F.i}$
I- None	DR<0.2	0	0	0	0	0	0
II- Slight	0.2<DR<0.5	0.5	0.9	0.5	3×10^{-5}	4×10^{-6}	1×10^{-5}
III- Light	0.5<DR<0.7	5	3.33	5	3×10^{-4}	4×10^{-5}	7.4×10^{-4}
IV-Moderate	0.7<DR<1.5	20	12.4	20	3×10^{-3}	3.2×10^{-2}	9×10^{-3}
V- Heavy	1.5<DR<2.5	45	34.8	45	2.6	0.35	0.09
VI- Major	2.5<DR<5.0	80	65.4	80	27	3.6	0.9
VII-Collapse	DR>5.0	100	100	100	35.7	35.7	18

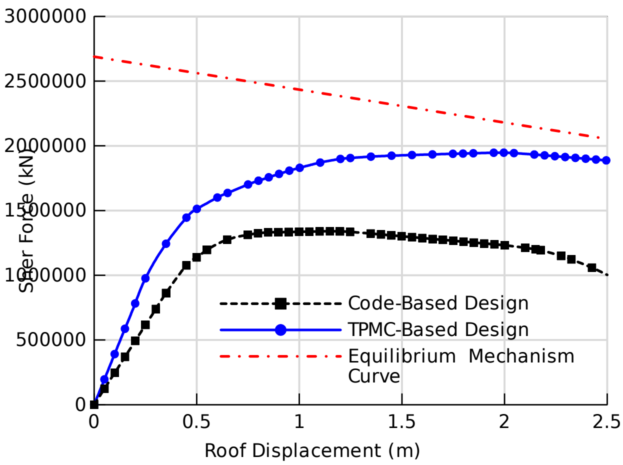

Figure 3. Pushover curves of designed structures. The Equilibrium Mechanism Curve is provided to show validation of design in TPMC procedure.

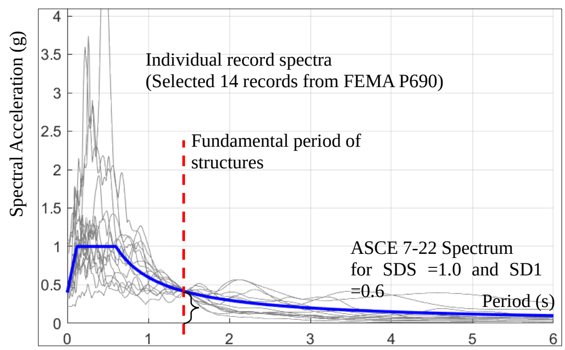

Figure 4. Scaled considered record spectra to standard spectrum at fundamental period used for IDA.

5. CONCLUSIONS

From an economic perspective within the realm of LCCA, TPMC emerges as the more favorable choice for the seismic design of structures.

Table 3. Record by record and total dispersion of and average spectral acceleration in each limit state.

LS	Code-Based Design			TPMC-Based Design		
	β_{RTR}	β_{TOT}	$\hat{S}_a$ (g)	β_{RTR}	β_{TOT}	$\hat{S}_a$ (g)
Slight	0.512	0.540	0.112	0.352	0.392	0.115
Light	0.513	0.542	0.281	0.352	0.393	0.288
Moderate	0.515	0.543	0.393	0.354	0.394	0.403
Heavy	0.519	0.547	0.836	0.346	0.387	0.868
Major	0.442	0.474	1.417	0.305	0.350	1.476
Collapse	0.339	0.381	2.251	0.275	0.325	2.316

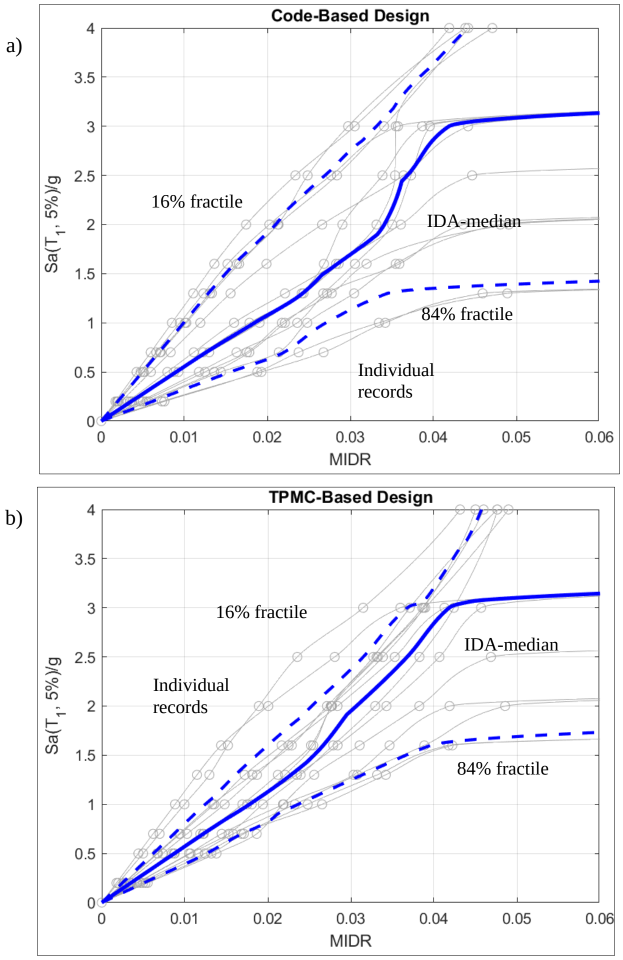

Figure 5. Maximum inter-story drift ratio (MIDR) of a) code-based design, and b) TPMC-based design SMF.

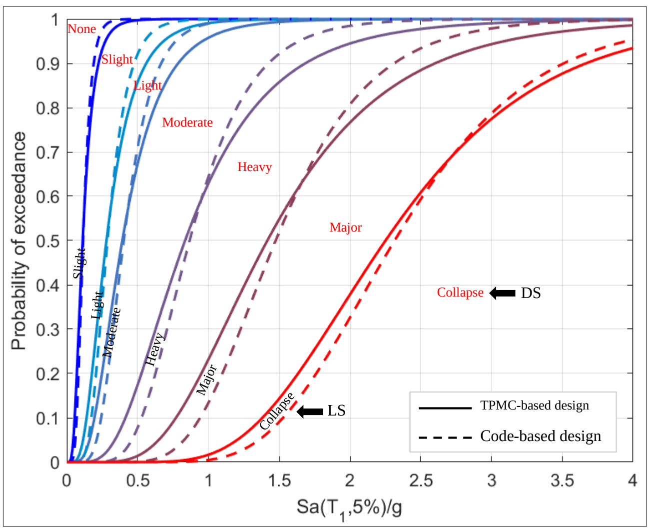

Figure 6. Fragility curves of designed SMFs for all limit states.

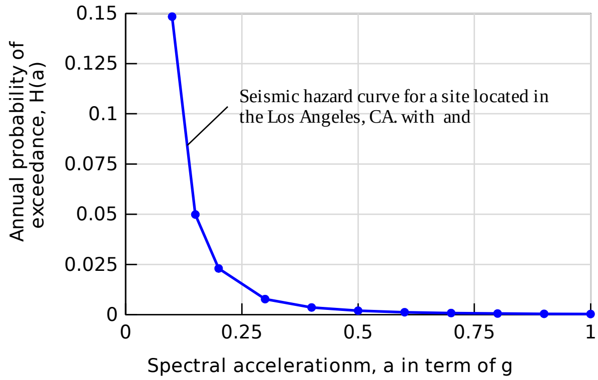

Figure 7. Considered seismic hazard curve used in LCCA.

Table 4. LCCA results for designed structures for $t=50$ years, $\nu=0.02$. and $\lambda=0.05$. The cost parameters are in term $1000.

DS	P_i	$C_{LC.i}$	C_{LC}	P_i	$C_{LC.i}$	C_{LC}
I- None	0.0	0.0	0.0	0.0	0.0	0.0
II- Slight	3.40E-01	118.6	14.8	1.82E-01	118.6	7.93
III- Light	1.75E-02	2118	13.6	9.68E-03	2118	7.53
IV- Moderate	1.03E-02	17600	66.3	5.79E-03	17600	37.44
V- Heavy	1.27E-03	146745	68.7	6.43E-04	146745	34.62
VI- Major	2.26E-04	1392562	115.5	1.28E-04	1392562	65.67
VII- Collapse	5.44E-05	26637210	532.2	4.21E-05	26637210	412.09
Total			**811.1**			**565.3**

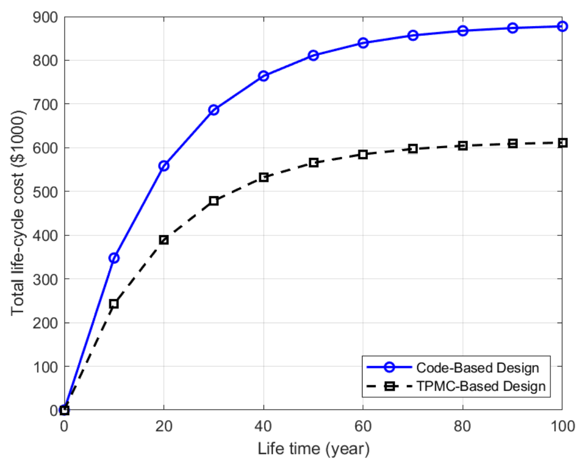

Figure 8. LCCA results vs. service life time of designed structures.

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7. AUTHOR BIOGRAPHIES

Mostafa Fathi Sepahvand holds a D. Eng. in Structural Engineering from Malayer University, Iran, and currently serves as a postdoctoral researcher at Chulalongkorn University. His research primarily centers around structural dynamics, nonlinear structural analysis, seismic design, plastic design, numerical methods in structural engineering, optimization, and probabilistic analysis of structures.

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