MODIFED SEISMIC RESPONSE COEFFICIENT (CS) FOR DESIGNING SUPER HIGH-RISE BUILDINGS USING PERFORMANCE-BASED DESIGN METHOD

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ABSTRACT

Performance-Based Design (PBD) method is widely used to design or evaluate super high-rise building against earthquake loads. The building is expected to present a certain level of performance set on FEMA 303 in response to ground motions, and should meet the target performance at Service Level Earthquake (SLE) and at Risk-Targeted Maximum Considered Earthquake (MCER). The performance level would be determined by using non-linear time history analysis and it requires non-linier parameter based on reinforcement of the structural elements. The common method proposed by Tall Building Initiative (TBI) requires the structural members are designed using response spectra at Service Level Earthquake (SLE). The problem is the ground motion and response spectra at Service Level Earthquake (SLE) are not always immediately available. In this paper, the modified seismic response coefficient (C_{S-M}) is introduced in designing the structural members, as an initial step of Performance-Based Design (PBD), using the common response spectra of Risk-Targeted Maximum Considered Earthquake (MCE_R) instead of Service Level Earthquake (SLE). The performance of buildings is evaluated at Service Level Earthquake (SLE) and Risk-Targeted Maximum Considered Earthquake (MCE_R) to validate that design with modified seismic response coefficient (C_{S-M}) is still in accordance with method by Tall Building Initiative (TBI).

Keywords: Modified Seismic Response Coefficient (C_{S-M}), Performance-Based Design (PBD), Risk-Targeted Maximum Considered Earthquake (MCE_R), Service-Level Earthquake (SLE), Tall Building Initiative (TBI)

INTRODUCTION

Super high-rise buildings must be designed as earthquake-resistant structures considering that Indonesia is a country that is susceptible of earthquakes. The higher a building, the lower the structural rigidity while the specified drift requirements must still be met. If only relying on dynamic analysis of spectrum response and using the value of seismic response coefficient (C_S) which is in accordance with the existing limits on the requirements of SNI 1726:2019, the design of the building will be wasteful because it is too conservative.

Then, further analysis is needed such as non-linear time history analysis to apply the Performance-Based Design (PBD) methods to optimize structure design and suitable with the desired performance. In this paper, the initial dimension and reinforcement data of structural members are required to be checked in ETABS and PERFORM-3D software.

At the end, the value of modified seismic response coefficient (C_{S-M}) of Risk-Targeted Maximum Considered Earthquake (MCE_R) that applied to the structure model which is representative to the Service-Level Earthquake (SLE) is obtained. It can be used practically at

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least as preliminary design because response spectra at Service Level Earthquake (SLE) is not always available and it takes time to do some research with site specific method in order to get the spectra data (SNI 8899:2020). So that, the modified seismic response coefficient (C_{S-M}) of Risk-Targeted Maximum Considered Earthquake (MCE_R) can be used as an alternative for designing the structure before the spectra data available.

STUDY OF LITERATURE

For non-linear time history analysis, the earthquake force applied to the structure is in the form of ground motion. The ground motion used as dynamic systems and applied Newmark-β as a direct integration method in numerical evaluation of the dynamic response of the structure. Therefore, the result will be more realistic than static analysis.

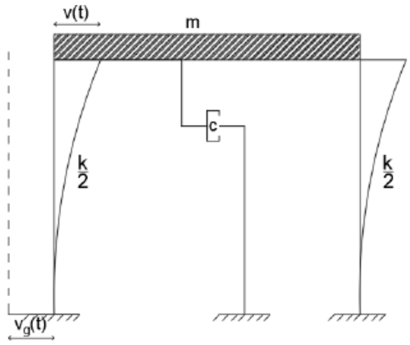


Figure 1. Newmark-β Illustration

$$m\ddot{v} + c\dot{v} + kv = P_{eff} \tag{1}$$

$$m\ddot{v} + c\dot{v} + kv = -m\ddot{v}_g \tag{2}$$

; $\ddot{v}$ is the acceleration, $\dot{v}$ is the velocity, v is the displacement, and $\ddot{v}_g$ is the ground acceleration.

Seismic response coefficient (C_S) is needed to calculate the design base shear based on static equivalent analysis. The formula is using the fundamental period, as follows.

$$C_{S-natural} = \frac{S_{D1}}{T(\frac{R}{I_e})} \tag{3}$$

The value of $C_{S\text{-natural}}$ should not be less than,

$$C_{S-minimum} = 0.044\, S_{DS} I_e \geq 0.01 \tag{4}$$

The modified seismic response coefficient (C_{S-M}) used if Risk-Targeted Maximum Considered Earthquake (MCE_R) given to the structure model is the average of the $C_{S\text{-natural}}$ and the $C_{S\text{-minimum}}$ which the limit set in SNI 1726:2019. Furthermore, the average seismic response coefficient values are varied in the range of multiplier factors from 0.8 to 1.2. All of the values should be bigger than 1.2 $C_{S\text{-natural}}$ (20% safety factor).

$$C_{S-M} = k\left\{\frac{C_{S-minimum} \pm C_{S-natural}}{2}\right\} \tag{5}$$

$$C_{S-M} = k\left\{\frac{0.044\, S_{DS} I_e + \frac{S_{D1}}{T(\frac{R}{Ie})}}{2}\right\} \tag{6}$$

$$0.8 \leq k \leq 1.2 \tag{7}$$

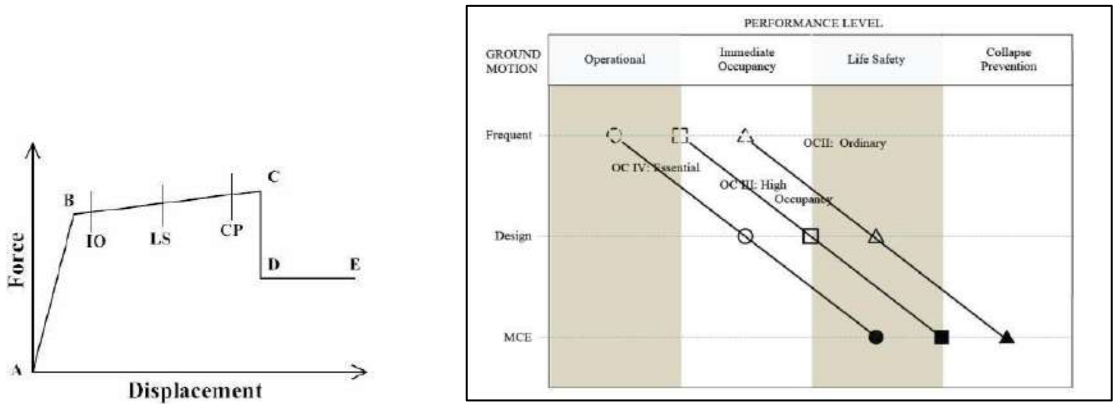
$$C_{S-M} \geq 1.2\,\frac{S_{D1}}{T(\frac{R}{Ie})} \tag{8}$$

$$T \geq 6\ seconds \tag{9}$$

S_{DS} = acceleration parameter of short-period design spectrum response

S_{D1} = acceleration parameters of 1-sec period design spectrum response
S_1 = acceleration parameters of the mapped design spectrum response
I_e = primacy factor of earthquake
R = response modification factors
T = fundamental period of structure (seconds)

For performance analysis, the structural elements should refer to this ATC-40 capacity curve and based on TBI:2017, the performance level of structure based on ground motion level is as follows.



(a) (b)

Figure 2. (a) Capacity Curve ATC-40, (b) Performance Level

For structure reliability analysis, the probability of collapse can be determined by using lognormal distribution. Parameters used in calculations are as follows.

$$\zeta = \sqrt{\ln(1 + \Omega_f^2)} \quad (10)$$

; ζ is standard deviation lognormal distribution

$$\Omega_f = \sqrt{\Omega^2 + 0.15^2 + 0.15^2 + \left(\frac{\Omega}{\sqrt{n}}\right)^2} \quad (11)$$

$$\Omega = \frac{\sigma}{\mu} \quad (12)$$

; n is amount of data, μ is mean values normal distribution, σ is standard deviation normal distribution.

$$\lambda = \ln \mu - \frac{1}{2}\zeta^2 \quad (13)$$

To ensure that the data obtained were lognormally distributed, a Kolmogorov-Smirnov validity test was performed with a significant level, α, with a target of 0.05. The Kolmogorov-Smirnov test is considered eligible if it meets the following equation.

$$D_n < D_n^a \quad (14)$$

The maximum value of D_n can be obtained from,

$$D_n = \max |F_n(x_i) - S_n(x_i)| \quad (15)$$

The D_n^a value is obtained by interpolation of the Kolmogorov-Smirnov table test.

Table 1. Kolmogorov-Smirnov Table Test

n \ α	0,20	0,10	0,05	0,01
5	0,45	0,51	0,56	0,67
10	0,32	0,37	0,41	0,49
15	0,27	0,30	0,34	0,40
20	0,23	0,26	0,29	0,36
25	0,21	0,24	0,27	0,32
30	0,19	0,22	0,24	0,29
35	0,18	0,20	0,23	0,27
40	0,17	0,19	0,21	0,25
45	0,16	0,18	0,20	0,24
50	0,15	0,17	0,19	0,23
n > 50	$\frac{1,07}{\sqrt{n}}$	$\frac{1,22}{\sqrt{n}}$	$\frac{1,36}{\sqrt{n}}$	$\frac{1,63}{\sqrt{n}}$

Then, fragility curve for lognormal distribution function is as follows.

$$f_{cap}(x_i) = \frac{1}{\xi\, x_i \sqrt{2\pi}} \exp\left[-\frac{1}{2}\left(\frac{\ln x_i - \lambda}{\xi}\right)^2\right] \quad (16)$$

METHODOLOGY

The analysis method used to evaluate the performance of the structure consists of non-linear time history analysis (by ETABS and PERFORM 3D) and structure reliability analysis by using Mathcad.

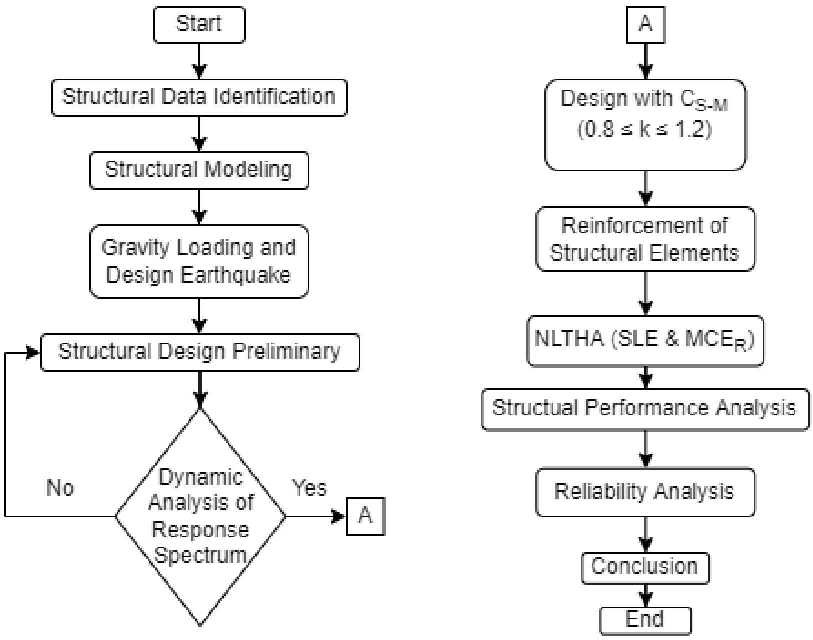


Figure 3. Methodology Flow Chart

This research methodology adapts the requirements set out in the code mentioned in the literature study.

ANALYSIS RESULTS

Structural design preliminary produces the fundamental period and seismic response coefficient calculations as shown in this following table.

Table 2. Calculation of Fundamental Periods and Seismic Response Coefficients

Tx	7.038	s	Ty	6.873	s
Ta min	2.9756	s	Ta min	2.9756	s
Ta max	4.1659	s	Ta max	4.1659	s
Cs max	0.0868		Cs max	0.0868	
Cs natural	0.0113	>0.01	Cs natural	0.0116	>0.01
Cs min	0.0267		Cs min	0.0267	
1.2 Cs natural	0.0136		1.2 Cs natural	0.0139	
C_{S-M} (k=0.8)	0.0190	>1.2 Cs natural	C_{S-M} (k=0.8)	0.0192	>1.2 Cs natural
C_{S-M} (k=0.9)	0.0152	>1.2 Cs natural	C_{S-M} (k=0.9)	0.0153	>1.2 Cs natural
C_{S-M} (k=1)	0.0171	>1.2 Cs natural	C_{S-M} (k=1)	0.0172	>1.2 Cs natural
C_{S-M} (k=1.1)	0.0209	>1.2 Cs natural	C_{S-M} (k=1.1)	0.0211	>1.2 Cs natural
C_{S-M} (k=1.2)	0.0228	>1.2 Cs natural	C_{S-M} (k=1.2)	0.0230	>1.2 Cs natural

Table 3. Information of Cs Curve vs T

Symbol	Cs Value		T (s)	Symbol	Cs Value		T (s)
	Cs min	0.026729	1.15		Cs min	0.026729	1.15
	Cs max	0.086783	0.45		Cs max	0.086783	0.45
	Cs natural	0.011315	7.038		Cs natural	0.011315	6.873
	C_{S-M} (k=0.8)	0.022827	1.65		C_{S-M} (k=0.8)	0.02299	1.55
	C_{S-M} (k=0.9)	0.020924	1.88		C_{S-M} (k=0.9)	0.021074	1.78
	C_{S-M} (k=1)	0.019022	2.02		C_{S-M} (k=1)	0.019158	1.92
	C_{S-M} (k=1.1)	0.01712	2.15		C_{S-M} (k=1.1)	0.017242	2.05
	C_{S-M} (k=1.2)	0.015218	2.51		C_{S-M} (k=1.2)	0.015326	2.41

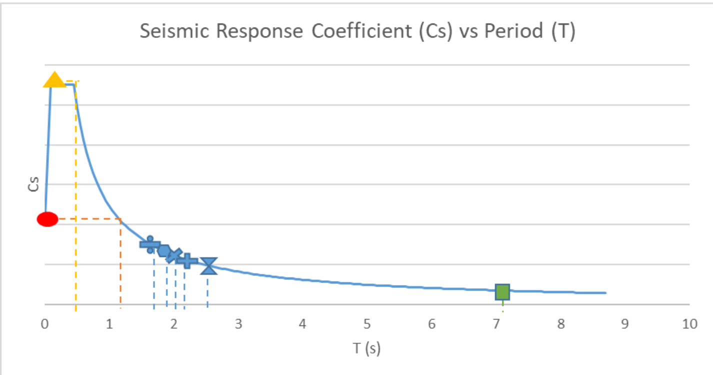


Figure 4. Illustration of Seismic Response Coefficient

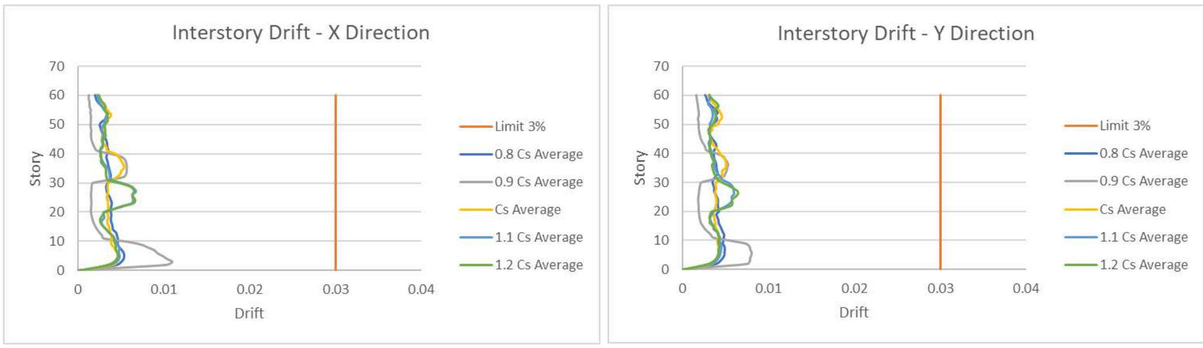
The figure above shows that the value of the modified seismic response coefficient (C_{S-M}) is between the $C_{S\text{-natural}}$ and the $C_{S\text{-minimum}}$.

Non-Linear Time History Analysis Result

The results consist of inter-story drift, residual drift, and plastic joint damage. Inter-story drift is a relative displacement between floors where the zero-point reference is below the observed floor.

Table 4. Inter-story Drift Requirements Based on SNI and TBI

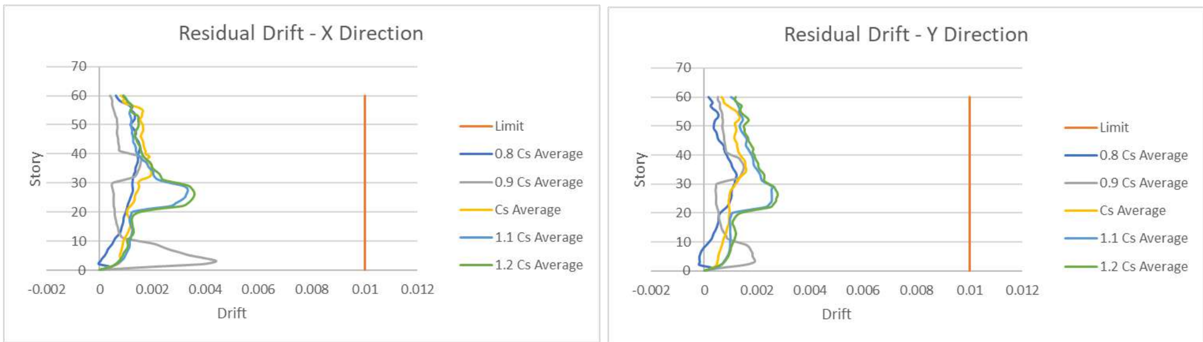
Earthquake Level	Lateral Drift Limit	
	SNI	TBI
MCE_R	2%	3%
SLE	-	0.5%



(a) (b)

Figure 5. Inter-story Drift: (a) X Direction, (b) Y Direction

Based on TBI section 6.7.3, for each floor, the absolute average value of the residual drift must not exceed 0.01. Residual drift is inter-story drift which occur at the end of the earthquake phase. These limits are intended to prevent excessive deformation in the aftermath of an earthquake which will cause a great danger to the surrounding construction in the event of a strong aftershock.



(a) (b)

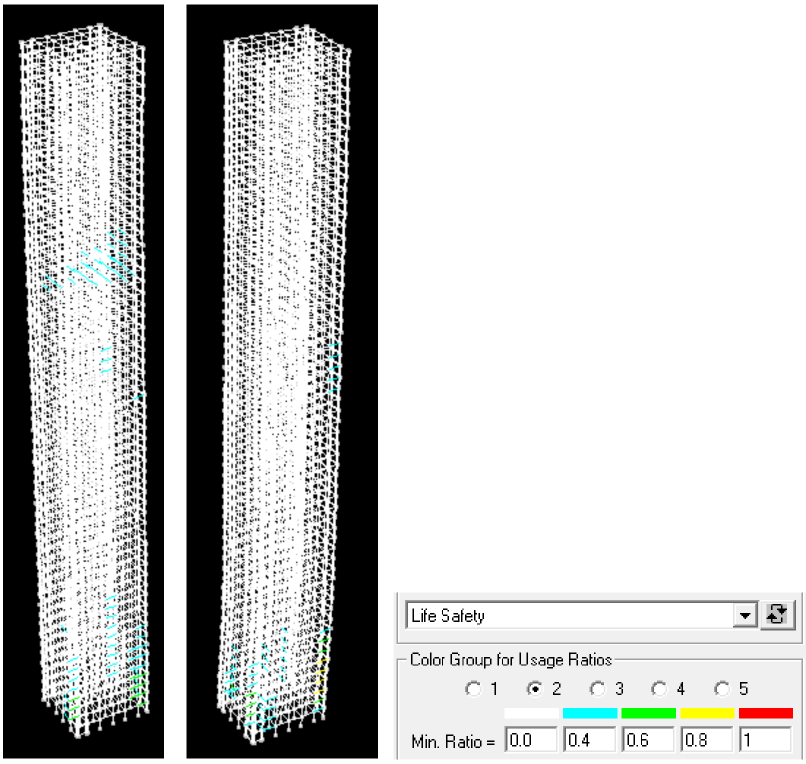
Figure 6. Residual Drift: (a) X Direction, (b) Y Direction

The figures above show that inter-story drift and residual drift do not exceed the required limits. Then, results summary of checking the damage to plastic joints in the final condition of each model after the earthquake force is given in the following table.

Table 5. Results of Plastic Joint Damage Checking

No	Case	Dir	Performance Level					
			SLE	0.8 Cs Average	0.9 Cs Average	Cs Average	1.1 Cs Average	1.2 Cs Average
1	"Kern County"	X	0.09147 IO	0.529 CP	0.505 CP	0.9752 LS	0.8615 LS	0.5205 LS
		Y	0.8779 Yield	0.542 CP	0.508 CP	0.9885 LS	0.796 LS	0.413 LS
2	"El Alamo"	X	0.6128 Yield	0.6487 CP	0.6277 CP	0.5208 CP	0.9981 LS	0.7184 LS
		Y	0.6607 Yield	0.6062 CP	0.5908 CP	0.57 CP	0.5578 CP	0.8544 LS
3	"Tabas_Iran"	X	0.3766 Yield	0.7804 CP	0.7696 CP	0.756 CP	0.655 CP	0.5999 CP
		Y	0.5179 Yield	0.9334 CP	0.8185 CP	0.7403 CP	0.732 CP	0.7266 CP
4	"Loma Prieta"	X	0.2997 IO	0.655 CP	0.6245 CP	0.5829 CP	0.5471 CP	0.5271 CP
		Y	0.3488 IO	0.6642 CP	0.6331 CP	0.5702 CP	0.5437 CP	0.9334 LS
5	"Kobe_ Japan"	X	0.4822 IO	0.9261 CP	0.8479 CP	0.783 CP	0.7576 CP	0.5554 CP
		Y	0.3933 IO	0.8846 CP	0.8264 CP	0.7597 CP	0.7298 CP	0.5554 CP
6	"Hector Mine"	X	0.5033 Yield	0.7163 CP	0.5806 CP	0.5576 CP	0.8929 LS	0.8536 LS
		Y	0.303 Yield	0.6766 CP	0.6638 CP	0.6368 CP	0.546 CP	0.9672 LS
7	"Duzce_Turkey"	X	0.4317 Yield	0.8623 CP	0.7321 CP	0.6534 CP	0.6253 CP	0.4769CP
		Y	0.01025 Yield	0.8716 CP	0.6705 CP	0.5911 CP	0.5813 CP	0.4594 CP
8	"Chuetsu-oki_ Japan"	X	0.5528 IO	1.05 CP	0.9259 CP	0.8165 CP	0.7781 CP	0.7525 CP
		Y	0.6162 IO	1.107 CP	1.058 CP	0.8371 CP	0.8086 CP	0.5645 CP
9	"Iwate_ Japan"	X	0.1508 IO	0.5505 CP	0.5283 CP	0.5273 CP	0.8527 LS	0.7135 LS
		Y	0.1518 IO	0.5295 CP	0.9964 LS	0.9769 LS	0.7958 LS	0.6258 LS
10	"El Mayor_ Mexico"	X	0.0069 Yield	0.5814 CP	0.5254 CP	0.9623 LS	0.858 LS	0.804 LS
		Y	0.7643 Yield	0.5389 CP	0.5347 CP	0.5197 CP	0.5139 CP	0.9722 LS
11	"Darfield_ New Zealand"	X	0.0008 Yield	0.7329 CP	0.5688 CP	0.5343 CP	0.5341 CP	0.9999 LS
		Y	0.0097 Yield	0.6879 CP	0.6014 CP	0.5762 CP	0.5616 CP	0.9735 LS

Hence, results summary of checking the damage meet the required performance level based on Figure 2b. The greatest damage of the structural elements when earthquakes applied is dominated by beams, then shear walls and columns.

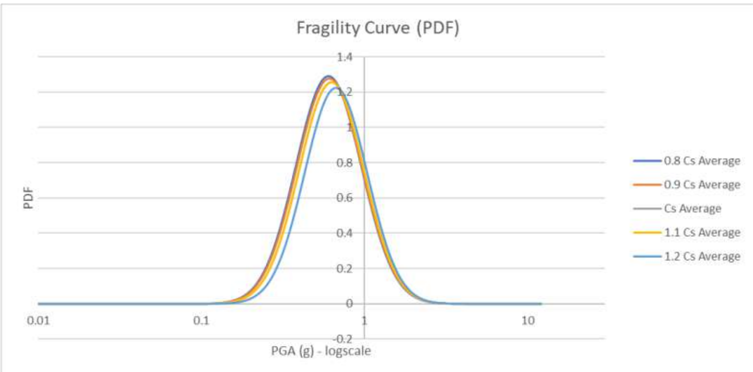


(a) (b) (c)

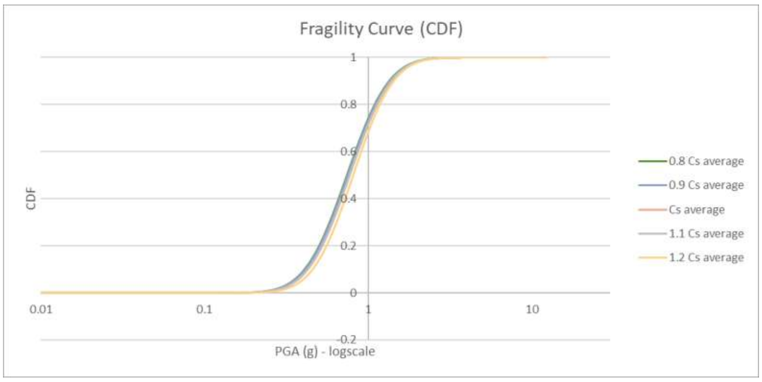
Figure 7. Final Condition of 1.2 Cs Average Model – El Alamo: (a) X Dir, (b) Y Dir, (c) Colour Group

Structure Reliability Analysis Result

For this analysis, PGA (earthquake acceleration) for each case will be scaling up incrementally until the structure is collapse. Then, the probability of collapse can be determined by using lognormal distribution as mentioned in study of literature. Based on the calculation, the lognormal distribution function fulfills the requirements of $D_n < D_n^{\alpha}$. The fragility curve for each structural model based on the lognormal distribution as shown the figure below.



(a)



(b)

Figure 8. Fragility Curve: (a) (PDF), (b) (CDF)

Then the risk of structure failure calculated by using risk integral method. The polynomial equation fitting with seismic hazard curve used is a level 6 equation as follows.

$$\ln NPGA = -4.2331 \times 10^{-5}(\ln PGA)^6 - 0.00189(\ln PGA)^5 - 0.0329(\ln PGA)^4 - 0.301(\ln PGA)^3 - 1.738(\ln PGA)^2 - 6.832(\ln PGA) - 13.3 \quad (17)$$

$$NPGA = P(PGA > x) \quad (18)$$

$$P(collapse) = \int_0^\infty P(PGA > x)\frac{dP[f_{cap}(PGA=x)]}{dx}dx \quad (19)$$

$$P(collapse\ in\ Y\ years) = 1 - [1 - P(collapse)]^Y \quad (20)$$

; Y = 50

Integration calculations are performed numerically using the Math-Cad application for each structural model, as follows.

$$f(x) = \frac{1}{\xi\, x\sqrt{2\pi}}\exp\left[-\frac{1}{2}\left(\frac{\ln x_i - \lambda}{\xi}\right)^2\right] \quad (21)$$

$$g(x) = P(PGA > x) = NPGA = \exp[-4.2331 \times 10^{-5}(\ln PGA)^6 - 0.00189(\ln PGA)^5 - 0.0329(\ln PGA)^4 - 0.301(\ln PGA)^3 - 1.738(\ln PGA)^2 - 6.832(\ln PGA) - 13.375 \quad (22)$$

$$P(collapse) = \int_0^\infty f(x)g(x)dx \quad (23)$$

The probability of collapse in 50 years is below 1% so it is an acceptable risk.

Table 6. Probability of Collapse (11 PGA Data)

Return Period	Cs modified				
	0.8 Cs ave	0.9 Cs ave	Cs ave	1.1 Cs ave	1.2 Cs ave
Annually	1.243×10^{-4}	1.198×10^{-4}	1.082×10^{-4}	9.044×10^{-5}	7.957×10^{-5}
50 Years	6.195×10^{-3}	5.973×10^{-3}	5.393×10^{-3}	4.512×10^{-3}	3.971×10^{-3}

CONCLUSION

Based on verification through non-linear time history analysis, in general, the performance of structures by applying frequent or Service-Level Earthquake (SLE) and Risk-Targeted Maximum Considered Earthquake (MCE_R) ground motions satisfy the requirements criteria of TBI 2017. So, there is a potential use of the modified seismic response coefficient (C_{S-M}). Although the results are quite promising, the modified seismic response coefficient (MSRC) is not intended to replace the current method of designing super high-rise buildings.

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