Proposal of Design Procedure for Seismic Isolation Codes Worldwide

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Abstract

After collecting information on building codes of Japan 2000, China 2010, USA IBC2009, Italy 2008 and Taiwan 2011 and using a benchmark building to compare among those codes, this paper proposes a preliminary design procedure for seismically isolated buildings referred as CW2012 (code of **C**IB **W**114). The CW2012 code mainly follows Japan 2000 code, since the number of seismically isolated buildings is most in Japan and buildings performed well under several great earthquakes where there were large input acceleration amplitudes or large displacements of the isolation system. However, in addition to the Japan 2000 code, several new aspects are introduced to cover other codes. An earthquake load having return period of 2,500 years is introduced to determine the isolation gap (building separations between the isolated structure and surrounding retaining walls or other fixed obstructions) and the test specifications of isolation system. A numerical coefficient related to the super-structure above the isolation system is introduced to maintain the design style using 50 years return period earthquake load of the super-structure such as China 2010 and Italy 2008 codes. Even though all of the codes include provisions for dynamic response analysis, the details required to undertake such an analysis for a seismically-isolated structure are not clearly available in any of the codes. Here, a procedure using time history analysis method to design seismically isolated buildings proposed by JSSI (2010a) is adopted.

Keywords: building code, seismically isolated building, earthquake load, equivalent linear analysis, time history analysis

1. INTRODUCTION

In the 1994 Northridge earthquake in the USA, the 1995 Hyogoken-Nanbu earthquake in Japan, the 1999 Chi-Chi earthquake in Taiwan, the 2008 Wenchuan earthquake in China, the 2009 L'Aquila earthquake in Italy and the 2011 Great East Japan earthquake, seismically

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isolated buildings have been reported perform well (Higashino, M., S. Okamoto, 2006; Saito, etc. 2011). Over the same period, building codes have been revised and updated to include requirements for design of seismically isolated buildings. Feng, *etc.* (2006) had a comparative report on building codes of Japan 2000, China 2001, USA IBC2003, Italy 2005 and Taiwan 2002 which was updated in 2010 (JSSI 2010b). In the USA, seismic isolation provisions have been included in building codes since first appearing in the 1991 Uniform Building Code. The current USA provisions are contained in the International Building Code (IBC 2009) which makes reference to the requirements of ASCE/SEI 7-05. In Japan, the most recent building code provisions took effect in 2000. In China and Taiwan it took effect in 2010 and 2011, respectively. In Italy, the new code took effect in 2008 over EURO 8. Unfortunately, in New Zealand, there is no specific code for seismic isolation, although the technology is well developed and numerous applications exist there.

In this paper, a preliminary design procedure CW2012 is proposed for seismically isolated buildings based on seismic isolation codes mentioned above. The CW2012 code mainly follows Japan 2000 code, since the number of seismically isolated buildings is most in Japan and buildings performed well under several great earthquakes where there were large input acceleration amplitudes or large displacements of the isolation system. However, in addition to the Japan 2000 code, several new aspects are introduced to cover other codes. An earthquake load having return period of 2,500 years is introduced to determine the isolation gap and the test specifications of isolation system. A numerical coefficient related to the super-structure above the isolation system is introduced to maintain the design style using 50 years return period earthquake load of the super-structure such as China 2010 and Italy 2008 codes. Even though all of the codes include provisions for dynamic response analysis, the details required to undertake such an analysis for a seismically-isolated structure are not clearly available in any of the codes. Here, a procedure using time history analysis method to design seismically isolated buildings proposed by JSSI (2010a) is adopted.

First, the concept of earthquake loads is summarized. A design procedure using equivalent linear analysis method or time history analysis method is described then.

2. DESIGN SPECTRUM

2.1 Earthquake load

In general, seismic load is expressed by 5% damping design spectrum as follows for all structures:

$$S_a(T) = Z\ I\ G_S(T)\ S_0(T) \tag{1}$$

Where, $Sa(T)$: the design spectrum on site, T: the fundamental period of the structure; Z: seismic hazard zone factor, I : the occupancy importance factor, $G_S(T)$: site class factor, and $S_0(T)$: the basic design spectrum.

The design spectrum generally consists of three parts, namely, an acceleration increasing portion in the extreme-short period, a uniform acceleration portion in the short-period range, and a uniform velocity portion in the longer-period range. A two-stage design philosophy is introduced generally in the code for designing an aseismic building. The two stages are usually defined as damage limitation (Level 1) and life safety limitation (Level 2). In the damage limitation stage, the structural safety performance should be preserved in the considered earthquake. In the life safety stage, the building should not collapse to assure the safety of human life.

In this paper, a different two-stage design philosophy is introduced for a seismically isolated building. A large earthquake with 10% probability of exceedance in 50 years (return period is about 475 years), which is used for designing a conventional building too, is defined to design the super-structure and sub-structure. For a RC frame system, the drift angle is proposed to be less than 0.005h (h: story height of a building) and 0.003h in the super-structure and sub-structure, respectively. An extreme large earthquake with 2% probability of exceedance in 50 years (return period is about 2500 years), is defined to obtain the maximum design displacement of the isolation system.

$$S_a(T)_{2500years} = \alpha\, S_a(T)_{475years} \tag{2}$$

Where, α = 1.3-1.5.

Table 1: Return period and story drift corresponding with each building code

	Level	Japan (2000)	China (2010)	USA (2009)	Italy (2008)	Taiwan (2011)	**CW2012**
Return period (Years)	Level 1	50[a]	50	—	50	—	
	Level 2	500[a]	—	UNKOWN	475	475	**475**
	Ex. Eq.[b]	—	1600-2500	2475	975	2500	**2500**
Story drift angle (RC Frame)	Level 1	1/200	1/550	—	1/200	—	
	Level 2	1/50a	1/50	1/50	None	1/50	

a: estimated; b: check the maximum design displacement of the isolation system

Table 2: Performance target of seismically isolated (SI) buildings

Return period (years)		EL1: 475 years	EL2: 2500 years
Super structure	Horizontal strength	Elastic limited[a]	
	Story drift angle	<1/200	
SI layer	Rubber isolator	γ<250% Tensile stress<1N/mm^2 within stable stress and deformation relation	Not Failure
	Sliding bearing	Design limit deformation	Not Failure
	Damper	Design limit deformation	Not Failure
Sub structure	Horizontal strength	Elastic	
	Story drift angle	<1/300	

a: Elastic limited state means the state which 1st hinge appeared in main structural member.

If one of earthquake load mentioned above is not defined in a country code, the relation shown in Eqn. (2) may be used. In accordance with the specific seismicity of each region, the return period of the considered seismic load differs considerably and is summarized in Table 1. Performance target of seismically isolated (SI) buildings corresponding the earthquake loads is shown in Table 2.

2.2 Long period earthquake load

In the Chinese code, the spectrum in the constant velocity portion is additionally increased to ensure the safety of structures having long natural periods, such as high-rise buildings or seismically isolated buildings. In the USA and Italian code, a constant displacement range is defined in the long period such as the earthquake load will be decreased in the long period. In the CW2012 code, we use same constant velocity portion with aseismic buildings for a SI building.

2.3 Damping coefficient

There is usually 20% critical damping in a seismically isolated building under EL1-475 earthquakes. As pointed by Feng (2006), the damping coefficient F_h (by which the spectrum at damping value other than 5% is calculated) defined in the Japanese code shown Eqn. (3) gave good accuracy of the equivalent linear analysis method (ELM), it is used in the CW2012 code.

$$F_h = \frac{1.5}{1+10(h_v+0.8h_d)} \tag{3}$$

Where, h_v : effective viscous damping of a fluid damper, h_d: a hysteretic damper decreased to 80% of the damping ratio which accounts for the effect by an earthquake comparing with stationary vibration. In Figure 1, typical spectral accelerations at 5% and 20% critical damping values are shown.

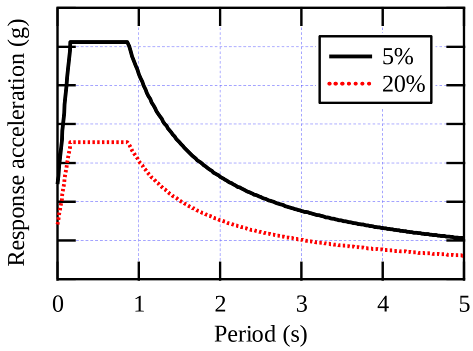

Figure 1: Typical design spectral acceleration at different critical damping values

3. DESIGN PEOCEDURE

3.1 Equivalent linear analysis

To design a SI building, both equivalent linear analysis and time history analysis method can be used. An equivalent linear analysis based on a single-degree-of-freedom (SDOF) system is used under limited conditions shown in Table 3. Almost same limitations proposed in the Japan 2000 code are used for CW2012.

Table 3: Limitations on the applicability of the equivalent linear analysis procedure

Structure \ Code	Japan	China	USA	Italy	Taiwan	**CW2012**
Limitation on site seismicity	-	-	$S_1 < 0.6g$	-	-	
Limitation on soil class	1,2	I,II,III	A,B,C,D	-	1,2	**1,2**
Maximum plan dimension	-	-	-	50m	-	
Maximum height of super-structure	60m	-	19.8m	20m	-	**60m**
Maximum number of stories	-	(H/b<4)	4	5	-	
Location of devices	Base only	-	-	-	-	**Base only**
Maximum mass-stiffness eccentricity	3%	-	regular	3%	regular	**3%**
K_v/K_e (Stiffness ratio between horizontal and vertical)	-	-	-	≥ 800	-	
Tension in isolator	Not allowed	1MPa	-	Not allowed	-	**Not allowed**
Yield strength	> 0.03W	-	-	-	-	**> 0.03W**
Period range of T_e	$T_2 > 2.5s$	-	$3T_f$ ~3.0s	$3T_f$ ~3.0s	≤ 2.5s	**$T_2 > 2.5s$**
Maximum value of T_v	-	-	-	< 0.1s	-	

g: gravity acceleration = 9.8m/s^2.
W: total weight above the isolation interface
T_f: natural period of the fixed-base super-structure.
T_2: period of the isolation system considering only the stiffness of rubber bearings.
T_e: equivalent period of the isolation system.
T_V: period of the isolation system in vertical direction.

3.1.1 Procedure of equivalent linear analysis method

In generally, the equivalent linear analysis method (ELM) can be illustrated as follows. The base shear force is obtained from the spectral acceleration and weight as shown in Eqn. (4).

$$\begin{gathered}\delta = \frac{M\, F_h(h,T_e) S_a(T_e)}{K_e} \\ Q_s = \gamma_e\, K_e\, \delta\end{gathered} \tag{4}$$

where,

δ: design displacement of the isolation system

M: total weight of the building

$F_h(h, T_e)$: damping coefficient;

h: damping ratio

$S_a(T_e)$ (g): site response acceleration considering site class

K_e: effective stiffness of the isolation system

γ_e: safety factor related to variation of properties with temperature, ageing or products tolerances discrepancy;

Q_s: shear force in the base of the super-structure;

The shear force, its distribution over the height of the super-structure and sub-structure are summarized in Table 4.

Table 4: The shear force of the super-structure and sub-structure and its distribution over the height

Structure	Symbol	**CW2012**
Isolation system	δ	$\delta = \dfrac{M\ F_h(h, T_e) S_a(T_e)}{K_e}$
	Q_s	$Q_s = \gamma_e\ K_e\ \delta$
Super-structure	Q_i	$Q_i = \dfrac{M_i\ H_i}{\sum_{i=1}^{n} M_j\ H_j} \dfrac{Q_s}{R_I}$
Sub-structure	Q_b	$Q_b = Q_s$

R_I: numerical coefficient related to the super-structure above the isolation system

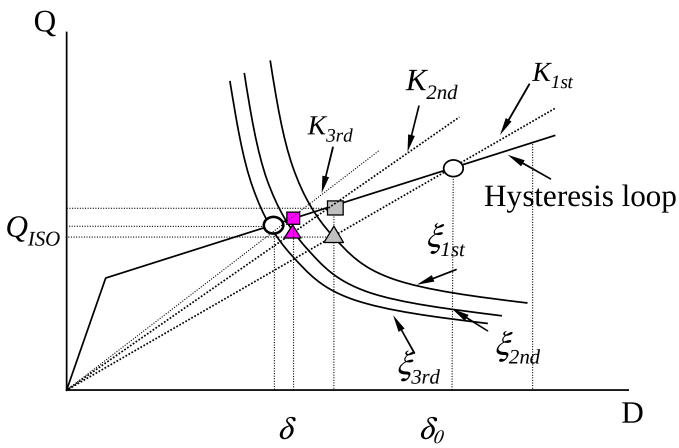

Figure 2: Illustration of the convergence procedure for the equivalent linear analysis method

To use ELM, calculation model must appropriately evaluate one mass for super-structure and characteristics of isolation devices at supposed response range. Modelling of isolation devices must appropriately evaluate stiffness and damping characteristics based on the test data by manufacturer. The convergence procedure of the equivalent linear analysis method is shown in Figure 2. The procedure is summarized as follows:

- Assume a displacement of the isolation system, δ_0.
- Calculate the effective stiffness, K_e, and damping ratio, ξ_e(h), of the isolation system, assuming a bi-linear model for the isolation system.
- Calculate the equivalent period, T_e, of the isolation system.
- Calculate the corresponding response reduction factor, $F_h(h, T_e)$, and the spectral acceleration, $S_a(T_e)$.
- Calculate a new isolation system displacement, δ, using Eqn. (4).
- Repeat the above steps until δ converges.

3.1.2 Maximum design displacement

The maximum displacement is obtained using same procedure shown in Eqn.(5) under EL2-2500 earthquake load.

$$\delta_M = \frac{M\,F_h(h, T_e) S_{a,2500}(T_e)}{K}$$
$$\delta_{rM} = \gamma_t \gamma_e \delta_M \tag{5}$$

Where,

δ_M: the maximum design displacement under EL2-2500 earthquake load;

δ_{rM}: the maximum design displacement used to determine the isolation cap and test specifications of isolation system;

γ_t: coefficient related to the eccentricity of the isolation system.

3.1.3 Others

Following items should be checked over for isolation system in the design procedure.

- The yield strength of the isolation system should be greater than the wind load.
- No tension is allowed in a rubber isolator at the design displacement.
- The isolation system should not collapse at the maximum design displacement.

3.2 Time history analysis procedure

Even though all of the codes include provisions for dynamic response analysis, the details required to undertake such an analysis for a seismically-isolated structure are not clearly available in any of the codes. In most of the codes two dynamic response analysis methods are defined: response spectrum analysis and time history analysis. For a seismically isolated building, the time history analysis method is the most accurate and is widely used.

In CW2012, a procedure using time history analysis method to design seismically isolated buildings proposed by JSSI (2010a) is adopted.

3.2.1 Input motions

In the time history analysis method, synthetic input motions that have been spectrally-matched with the design response spectrum or real earthquake records appropriately scaled or modified should be used for the dynamic response analyses. Since results from the dynamic response analyses are strongly dependent on the selected input motions, several input motions are recommended. In the Japanese code, based on more than three (usually six) input motions, the maximum response values are taken as design values. In the Chinese code, based on three input motions, the average response values are taken as design values. In the USA and Italian codes, a minimum of three time history pairs must be used for the analyses. If three time history pairs are used, the design must be based on the maximum response quantities obtained, however, if seven (or more) time history pairs are used the design may be based on the average values of the calculated responses. Since the time history analysis method usually results in smaller response values than those by the equivalent linear analysis method, in the USA and Taiwan codes the results of the time history analyses are limited by the results from the equivalent linear analysis method. For example, in the USA code, the total design displacement of the isolation system shall not be taken as less than 90% of the result due to the equivalent linear analysis method. On the other hand, there is no limitation in the Japanese and Italian codes.

In CW2012, seven (or more) time history pairs are used, thus the design is directly based on the average values of the calculated responses. If vertical spectral acceleration is not defined in the code, ratio between vertical spectrum and horizontal spectrum defined in BRI&BCJ (1992) may be used. The degree of compatibility of the synthetic input motion with the design spectrum is defined by the following four parameters:

- The ratio of the input motion response spectrum $\boldsymbol{S_{psv}}(\boldsymbol{T_i})$ to the design spectrum $\boldsymbol{DS_{psv}}(\boldsymbol{T_i})$ should not be less than 0.85.

$$(\varepsilon_i)_{\min} = \left(\frac{S_{psv}(T_i)}{DS_{psv}(T_i)}\right)_{\min} \geq 0.85$$

- The coefficient of variation (ν:COV) of the input motion response spectrum should be less than 0.05. *N* is total point number accounted.

$$\nu = \sqrt{\frac{\sum(\varepsilon_i - 1.0)^2}{N}} \leq 0.05$$

- The total average value of the input motion response spectrum should be larger enough.

$$\left|1-\varepsilon_{ave}\right| = \left|1-\frac{\sum \varepsilon_i}{N}\right| \leq 0.02$$

- The spectral ratio at long period range (say one to five second for example) should be larger than 1.0.

$$SI_{ratio} = \frac{\int_1^5 S_{psv}(T)dT}{\int_1^5 DS_{psv}(T)dT} \geq 1.0$$

3.2.2 Analysis model

Both super-structure elements and isolation system should be modelled properly in the estimated response range. The isolation system should be modelled based on the test results especially for the characteristics of stiffness and hysteresis.

A simple shear-rocking system having multiple-degree-of-freedom is enough to understand the fundamental characteristics of seismically isolated buildings. The super-structure is modelled as a non-linear shear type system, where the shear elements are usually derived from a static non-linear push-over analysis. However, in case of bending deformation is dominated such as high rise buildings, a bending shear system is preferable. The deformation of the super-structure is smaller than the conventional aseismic building, thus the real damping ratio may be smaller. A stiffness proportional damping matrix is generally used in the response analysis. The damping of the super-structure should be determined based on the first mode of the fixed model. In the case of an intermediate story isolation building, damping coefficients may be determined separately for the super-structure or the sub-structure by similar consideration. The isolation level is modelled as a shear-rocking system, where a bilinear model is used for the shear component. The viscous damping of shear property of a rubber bearing or a lead rubber bearing may be neglected. The elastic rocking component is calculated from the vertical stiffness of the bearings. A horizontal input motion is applied directly at the base.

To obtain the overturning or uplift forces on individual isolators, or the configuration of super-structure or isolation system is not regular, or the torsional vibration is dominated, a three-dimensional model may be used. Since the model becomes more complex, the design should be careful on the modelling of total system. The vertical stiffness of a rubber bearing should be modelled as non-linear to obtain uplift response correctly. One should be careful on the damping matrix also. If stiffness proportional damping matrix is used, the damping in the vertical direction may be overestimated. Three dimensional input motions may be applied all at once. The results may be superposed together after horizontal and vertical analyses separately too.

There developed so many kinds of isolation devices, such as rubber isolators, sliding bearings, steel dampers and oil dampers etc. Analysis models should be determined based on experimental results carefully. The limitation of each model should be taken care also. For instance, hardening is observed in the large shear deformation of a rubber bearing and should be modelled properly in a large deformation response.

Variation of properties with temperature, ageing or products tolerances discrepancy of isolators or dampers should be included in the modelling. If the torsional vibration is dominated, two horizontal direction properties of the isolation system should be modelled properly.

3.2.3 Analysis results

The shear force and story drift angle of the super-structure and sub-structure should be under the capacity of the elements. To keep the function of equipment, the response acceleration is also to be checked. The maximum horizontal displacement of the isolation system should be less than the design allowable displacement. The vertical displacement should be less than the allowable clearance too, especially for the system having large vertical deformation such as friction pendulum bearing system. Torsional vibration may cause larger deformation of outer isolators.

If the deformation of a rubber isolator becomes large, the allowable vertical pressure will become small as shown in Figure 3. If tension occurred in a rubber isolator, the vertical load in all isolators will be re-allocated, which should be checked over by summation by vertical input motion.

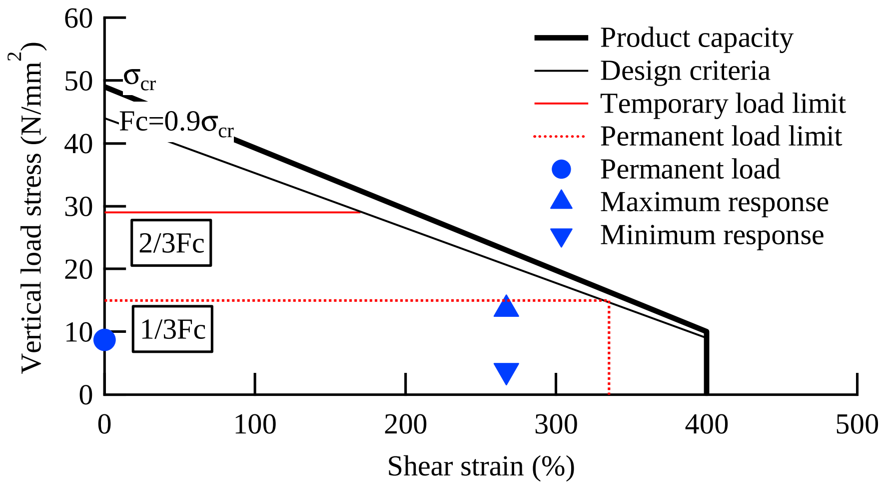

Figure 3: Checking of vertical load with shear strain for a rubber isolator

To understand the analysis results, it is important to confirm how much the input earthquake energy was dissipated by the super-structure and isolation system. The dampers should have enough capacity to dissipate almost all input earthquake energy even in the long duration earthquake such as the 2011 Great East Japan earthquake. The dissipated energy by the super-structure is smaller, the performance of the isolated building is better.

4. CONCLUSIONS

A preliminary design procedure CW2012 was proposed for seismically isolated buildings based on seismic isolation codes worldwide. The CW2012 code mainly follows Japan 2000 code. However, in addition to the Japan 2000 code, several new aspects are introduced to cover other codes. An earthquake load having return period of 2,500 years is introduced to determine the isolation gap and the test specifications of isolation system. A numerical coefficient related to the super-structure above the isolation system is introduced to maintain the design style using 50 years return period earthquake load in some codes. A procedure using time history analysis method to design seismically isolated buildings proposed by JSSI (2010a) is adopted.

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