

# Design of base-isolated buildings: An overview of international codes

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**ABSTRACT:** Base isolation is arguably the most reliable method for providing enhanced protection of buildings against earthquake-induced actions, by virtue of a physical separation between the structure and the ground through elements with limited force capacity and (often) enhanced energy dissipation. Such a design solution has shown its effectiveness in protecting both the structural and non-structural components, hence preserving their functionality even in the aftermath of a major seismic event.

Despite lead rubber bearings being invented in New Zealand almost forty years ago, the Christchurch Women's hospital was the only isolated building in Christchurch when the Canterbury earthquakes occurred in 2010/11. Furthermore, a reference code for designing base-isolated buildings in New Zealand is still missing. The absence of a design standard or at least consensus on design guidelines is a potential source for a lack of uniformity in terms of performance criteria and compliance design approaches. It may also limit more widespread use of the technology in New Zealand.

The present paper provides an overview of the major international codes (American, Japanese and European) for the design of base-isolated buildings. The design performance requirements, the analysis procedures, the design review process and approval/quality control of devices outlined in each code are discussed and their respective pros and cons are compared through a design application on a benchmark building in New Zealand. The results gathered from this comparison are intended to set the basis for the development of guidelines specific for the New Zealand environment.

## 1 INTRODUCTION

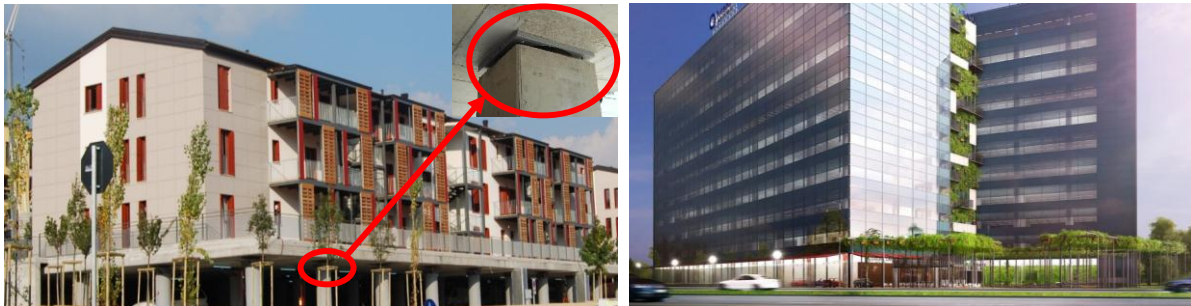
The primary objective of the paper is to provide an overview of the major international codes for designing base-isolated buildings, following the example of previous studies performed in Japan (as summarised amongst all in Feng et al., 2006), in order to compare their respective provisions and assess their potential for implementation in New Zealand. The focus is on design procedures, code requirements and structural performance criteria. The three codes addressed in this paper are the Japanese (Building Standard Law), the American (ASCE 7-10) and the European (Eurocode 8, EC8) codes. An example of base isolated buildings in these respective countries is shown in Figure 1, Figure 2 and Figure 3. A case study of a building in New Zealand is presented to compare key provisions of the different codes. The comparison focuses on the implementation of the Equivalent Lateral Force Method (ELFM).



**Figure 1. Example of base isolated buildings in Japan: Takeda Pharmaceutical Company Shonan Research Center, Fujisawa City, Kanagawa**



**Figure 2. Example of base isolated buildings in the US: Cathedral of Our Lady of Los Angeles, Los Angeles, CA (left), Municipal Service building, Glendale, CA (right)**



**Figure 3. Example of base isolated buildings in Europe: L'Aquila, Italy (left), Bucharest, Romania (will be the first in the country) (right)**

## 2 INTERNATIONAL CODES

### 2.1 Japanese code (BSLEO, 2000)

The regulation framework in Japan is articulated in a Building Standard Law (the last revision was in 1998 and it introduced performance-based design concepts) and associated Enforcement Order (Building Standard Law Enforcement Order [BSLEO], 2000), required for the Law to be effective. Technical standards for construction specifications and structural calculations methods are then outlined by the Minister of Land, Infrastructure and Transport (MOLIT) in the form of Notifications. The design of base isolated buildings is governed by Notification No.1457 and No.2009, while the isolation devices are governed by Notification No.1446, issued by MOLIT on October 17, 2000 (Kani & Otani, 2002). At the end of 2010, it is estimated that 3,000 buildings and 5,000 detached houses were using seismically isolated technology.

The construction and structural calculation of seismically isolated buildings can be carried out following one of the three routes summarized below, and discussed in more detail in a separate section.

- Route 1: No structural calculation for “small constructions” [actually never used in practical applications].
- Route 2: Notification 2009 route for “normal constructions” (ELFM) [yet very rarely adopted in practical application, with an estimated 10%].
- Route 3: Non-Linear Time-History Analyses, (NLTHA) for special construction [the most widely adopted approach, covering an estimated 90% of practical applications].

### 2.1.1 Design spectrum

The design spectrum is defined based on two intensity levels (Table 1) corresponding respectively to a serviceability (L1) and a life safety (L2) limit state (the latter will be presented as Design Earthquake (DE) level for consistency in the text).

It is worth noting, even if outside the scope of the paper that the Japanese code uses a return period of 45-50 years for the Serviceability limit state (SLS) while New Zealand was and is still using (outside the Canterbury earthquake region) a SLS earthquake with a return period of 1/25 (25 years). Only the new risk factor for the serviceability limit state ( $R_s = 0.33$ ) in the Canterbury earthquake region has raised that to become 1/50 but it has not been explained clearly that it is not only due to the higher short-term seismicity but it is actually a major issue of compatibility with what overseas is used as SLS level.

**Table 1. Earthquake levels and performance criteria – Japanese code (Kani et al., 2010)**

Intensity level	L1 (damage limitation) ~ SLS	L2 (life safety) ~ DE
Return period (years)*	50	500
Superstructure performance target	Elastic ( $\theta < 1/500$ )	Elastic limited ( $\theta < 1/300$ )
Substructure performance target	Elastic ( $\theta < 1/1000$ )	Elastic limited ( $\theta < 1/500$ )
Isolators performance target	$\gamma < 1 \div 1.5$	$\gamma < 1.5 \div 2.5$ $\sigma_{\text{tension}} < 0$ (No tension) (Route 3 only: $\sigma_{\text{tension}} < 1\text{MPa}$ in rubber bearings and tension allowed in rail sliders)
* estimated; $\theta$ : Interstorey drift; $\sigma_{\text{tension}}$ : Tensile strain in isolators; $\gamma$ : Shear strain in isolators;		

The Japanese code, refers to a 5%-damped spectral acceleration by Equation 1 defined below:

$$S_a(T) = ZG(T)S_0(T) \quad (1)$$

where  $Z$  is the seismic hazard zone factor (which varies between 0.7 to 1.0 in Japan),  $G_s(T)$  is the soil amplification factor and  $S_0(T)$  is the design acceleration at bedrock (Shear wave velocity,  $V_s > 400\text{m/s}$ ).

The soil amplification factor can be calculated based on the local properties of the soil layers or from code-defined values for three different soil classes (Higashino and Okamoto, 2006). Sample design spectral shapes are reported in Figure 4. In engineering practice, the  $G_s(T)$  is usually calculated iteratively based on the investigated  $V_s$  or  $N$  values and types for the soil profile rather than directly using the coefficients defined in the code in Route 2. The displacement spectra, actually a pseudo-displacement spectra, do not feature a corner period for the transition to a constant displacement demand for long-period structures.

For NLTHA, the code allows the adoption of either artificially simulated or natural records, scaled to match the design spectrum. The minimum number of records to consider is 3, with the common

practice considering 6 records. The design is based on the maximum response values. Amongst the most commonly used records is the so called “golden set”, defined by the El Centro NS 1940, Taft EW 1952 and Hachinohe NS 1968 records (the latter well known for its long period peak response), simply scaled by a factor to obtain a peak velocity of 0.5m/s for Level 2 input, and 0.25m/s for Level 1. In addition to the standard ground motions, a suite of three regionally specific motions is developed, either using artificially simulated or natural records, scaled (in the frequency domain, (Higashino and Okamoto, 2004)) to match the design spectrum. Usually, the design set includes the JMA Kobe record (Becker et al., 2010). The compatibility criteria with the design spectrum are typically defined as summarised in Table 2, although this set of criteria does not represent a code requirement but rather “common practice” only. For the input ground motion, a time history analysis method using equivalent linear analysis (e.g. SHAKE) or a non-linear Ramberg-Osgood model are usually used to obtain  $G_s(T)$ . The design spectrum is defined as the dominant component.

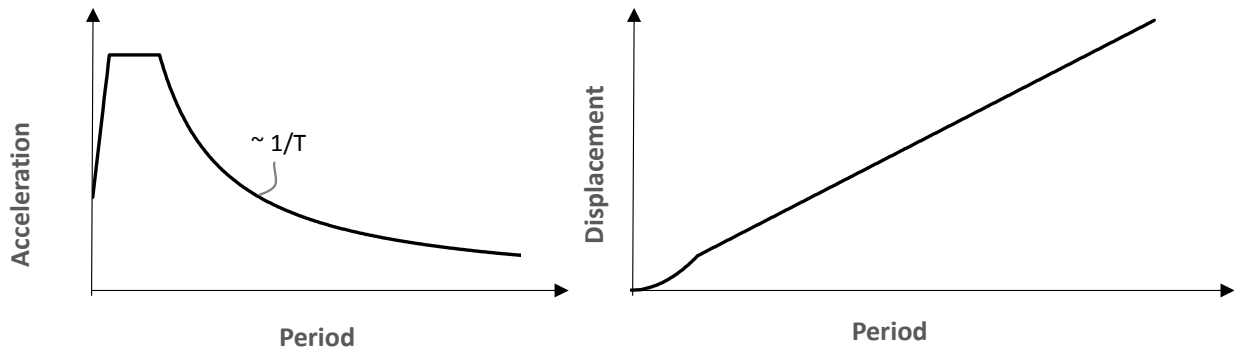


Figure 4. Japanese code DE 5%-damped spectral shape: acceleration (left), displacement ordinates (right)

Table 2. Earthquake records scaling criteria – Japanese “design practice”

Criteria	Note	General Remarks
$\varepsilon_{\min} = \left( S_{psv}(T_i) / DS_{psv}(T_i) \right)_{\min} \geq 0.85$	Min value $\geq 85\%$ target	Scaling is referred to the pseudo velocity ordinates.
$COV \leq 0.05$	Low dispersion	
$ 1 - \varepsilon_{ave}  \leq 0.02$	Average value close to target	Consideration of the full period range (even if not clearly specified)
$SI_{ratio} = \left( \frac{\int_1^5 S_{psv}(T) dT}{\int_1^5 DS_{psv}(T) dT} \right) \geq 1.0$	Housner spectrum intensity calculated over the long-period range (1-5s)	
$S_{psv}$ : pseudo-velocity response spectrum ordinate $DS_{psv}$ : pseudo-velocity design response spectrum ordinate $\varepsilon = \left( S_{psv}(T_i) / DS_{psv}(T_i) \right)$ COV: coefficient of variation of the single record with respect to the design spectrum $SI_{ratio}$ : spectral ratio at long-period range		

The design code in Japan does not include any importance factor (IF) for essential/critical facilities or buildings where people are likely to congregate. However, it is common practice for local authorities to require an importance factor to be adopted in practical applications. Typical values are 1.25 for public buildings like schools and 1.5 for essential facilities, like for example hospitals. Such IFs amplify the design spectrum, hence effectively requiring the design for a larger return period event, in order to achieve higher performance in buildings with IF larger than 1.0 when compared to normal class building for the same earthquake intensity.

2.1.2 *Design method*

Route 1: No structural calculation required. This route is meant to be available only for “small constructions” (as defined in Table 3) and the design can be developed satisfying a set of boundaries (Table 4), but without any calculation. This design pattern is presented here for completeness, however, based on the authors’ experience, there is no evidence of any real implementation of this approach in Japan.

**Table 3. Definition of “small constructions” – Japanese code**

<b>Scenario</b>	<b>Requirement</b>	
1	total floor area	$\leq 100 \text{ m}^2$
2	Timber construction	not more than two stories or not more than $500 \text{ m}^2$ total floor area and not more than 13 m in height
3	Buildings, other than timber construction	single-story or of total floor area not more than $200 \text{ m}^2$

**Table 4. Requirements for the application of design Route 1 – Japanese code**

Parameter	Requirement		Comments
[total floor area x storey] / [# isolators]	$\leq 15 \text{ m}^2$		To provide redundancy and redistribution of vertical loads
[total yield strength of the isolation system ( $F_y$ ) of isolators] / [BLD floor area (1 level)]	0.22 ÷ 0.36	Lightweight buildings (1 storey)	Min: to ensure stability under non-seismic loading. Max: to limit the shear demand in the building.  Building weight: allow for larger shear forces to force the adoption of units with adequate stiffness to ensure stability.
	0.29 ÷ 0.49	Lightweight buildings (2 storeys)	
	0.34 ÷ 0.58	Other buildings	
[total strength of the isolation system at the design displacement ( $F_d$ ) of isolators] / [BLD floor area (1 level)]	0.72 ÷ 1.09	Lightweight buildings (1 storey)	
	0.98 ÷ 1.47	Lightweight buildings (2 storeys)	
	1.17 ÷ 1.75	Other buildings	
Displacement capacity of isolators	$\geq 350\text{mm}$		
Equivalent damping at design displacement	$\geq 20\%$		
Ensure appropriate load transfer from the superstructure	Locate isolators at the base of columns/walls		
Minimum tangent period	2.5s (2.0s for building height < 13m)		Isolated period referred to the stiffness at the design displacement level (Kani & Otani, 2002)
Axial force on isolators	No tensile forces		Account for 0.3g up/down static vertical seismic acceleration (Kani et al., 2010)
Maximum mass-stiffness eccentricity	3%		

**Route 2:** Notification 2009 route for normal constructions. Under this path, the building can be designed on the basis of a 2D linear equivalent static analysis, subjected to the approval a building official. The code defines as “normal constructions”:

- Buildings of total height (H) not greater than 60m (approximately 15-20 storey buildings),

- local soil class 1 or 2 (rock or firm ground conditions), without liquefaction potential.
- isolators must be located at foundation level (i.e. a podium or car parking at the isolated level is not possible).

Under this route, the design should be performed following the ELMF, provided that the conditions outlined in Table 5 are satisfied. The ELMF allows for the definition of the design displacement ( $\Delta_{ELFM}$ ), and associated forces, based on the effective stiffness and damping of the isolators (IS), as summarised in section 3, with reference to a case study.

**Table 5. Requirements for the application of the ELMF (Route 2) – Japanese code**

Parameter	Requirement	Comments
Building height (max)	60m	
Soil conditions	Rock or firm ground	
[total $F_y$ of isolators] / [total BLD seismic weight]	> 0.03	Min: to ensure stability under non-seismic loading.
Ensure appropriate load transfer from the superstructure	Locate isolators at the base of columns/walls	
Minimum tangent period	2.5s	Isolated period referred to the stiffness at the design displacement level (Kani & Otani, 2002)
Axial force on isolators	No tensile forces	Account for 0.3g up/down static vertical seismic acceleration (Kani et al., 2010)

**Route 3:** Other route for special construction. For buildings other than “small” or “normal” structures, the design should be performed through NLTHA, and specifications and calculations should be approved by MOLIT. This remains the most widely adopted route in Japan, and it covers approximately 90% of base isolation designs.

### 2.1.3 Structural modelling

Where the ELMF is used, the contributions of the energy dissipation provided by the isolators is considered through a damping reduction factor  $R$  ( $F_h$  in the code) which multiplies the spectral ordinates to obtain the design values (Equation 2, Figure 8 (a)).

$$1.0 \geq F_h = \frac{1.5}{1 + 10 \left( \xi_v + 0.8 \xi_{eff,d} \right)} \geq 0.4 \quad (2)$$

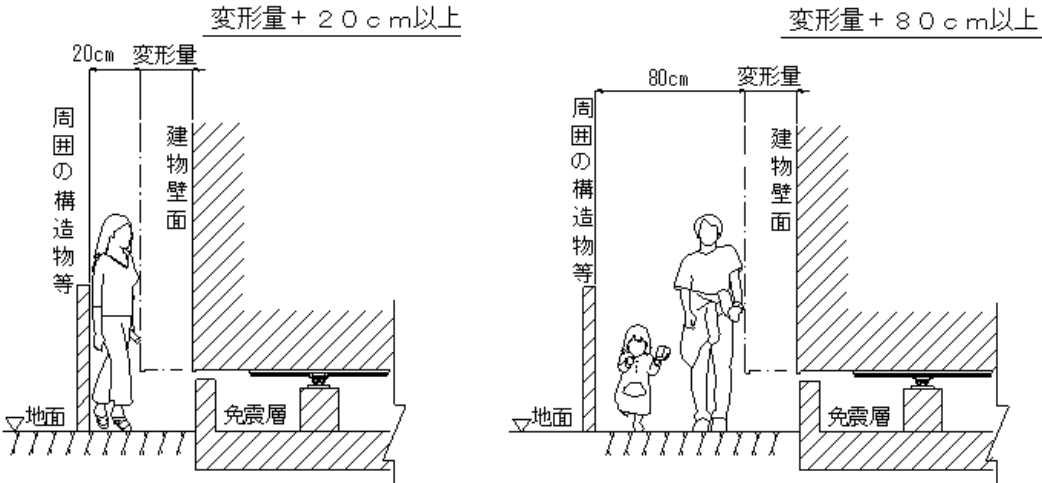
The damping reduction factor is estimated from the assessed damping coefficient  $\xi_v$  ( $h_v$  in the code) developed from pure viscous-type behaviour (does not cover structural damping) and the area-based equivalent damping coefficient  $\xi_{eff,d}$  ( $h_d$  in the code) provided by seismic isolators and hysteretic devices in the seismic isolation layer. Note the reduction of the hysteretic contribution.

For NLTHA, the superstructure is modelled as a non-linear shear-type system, where the shear-type elements are usually derived on the basis of a static non-linear pushover analysis. Equivalent seismic masses are applied at floor levels and the isolation system is modelled through a bi-linear shear spring and a linear rocking spring (Higashino and Okamoto, 2006). The result is thus a very simple equivalent Multi-Degree-of-Freedom column model.

### 2.1.4 Structural performance requirements

With reference to the ELMF (Route 2), in addition to what already specified in Table 5, the code

requires the conditions summarised in Table 6 to be satisfied. Note the code does limit the analysis to L2 design level only in Route 2 (normal constructions). When the NLTHA method is adopted (Route 3, special constructions), there are no pre-defined code limits. Note the safety factor ( $\beta$ ) applied to the isolator's capacity. This can somehow be considered as "in lieu" of the collapse prevention limit state or MCE-type event as used or implied in other international codes as later discussed. In addition, it is very important to observe that the Japanese code introduces an additional safety factor (1.3) to amplify the level of design forces for the superstructure, in order to account for change of characteristics, environmental change and decay with time of the devices, if not otherwise evaluated. The code does also specify clearance requirements with respect to adjacent buildings, as defined in Table 6 and graphically represented in Figure 5.



**Figure 5. Required clearance around the perimeter of the building: for maintenance only (left), and when the area is used as general public passage/walkway for non-residents (right)**  
[\[http://www.menshin.biz/?q=node/351\]](http://www.menshin.biz/?q=node/351)



**Table 6. Structural performance requirements for the application of the ELM – Japanese code**

Limit state	L2
Isolators (IS)	<p><math>\gamma &lt; 1.5 \div 2.5</math></p> <p><math>\gamma</math> design shear strain in elastomeric bearings</p> <p>Design displacement: <math>\Delta_d &lt; \beta \Delta_u</math> where:</p> <p><math>\Delta_d = 1.1 \times 1.2 \times \Delta_{ELFM}</math> [1.1: for torsion / 1.2 to account for environmental effects and dispersion in properties]</p> <p><math>\beta</math> safety factor (elastic Isolation Device (ID) = 0.8 – sliding/friction device = 0.9 – Viscous Damper (VD) = 1)</p> <p><math>\Delta_u</math> = displacement capacity</p> <p>Design all connections for peak axial load &amp; <math>[1.2 \times \Delta_{ELFM}]</math></p> <p>Clearance <math>\geq</math> max of</p> <p>* <math>1.25 \Delta_d = 1.25 \times 1.1 \times 1.2 \times \Delta_{ELFM}</math></p> <p>* <math>200\text{mm} + \Delta_d = 200\text{mm} + 1.1 \times 1.2 \times \Delta_{ELFM}</math></p> <p>+ further 600mm if the area between buildings is used as the general public passage/walkway (Figure 5)</p> <p>Ensure possibility to replace/inspect ID</p> <p>Provide maintenance plan</p> <p>Provide fire protection: the cover should remain in place up to the design displacement level</p> <p>A sign must be posted at the location to allow visitors to observe that the building implements Base Isolation (BI)</p>
Superstructure (BLD)	<p>Elastic limited:</p> <p><math>\theta &lt; 1/200 - H &lt; 13\text{m}</math></p> <p><math>\theta &lt; 1/300 - H \geq 13\text{m}</math></p> <p>Design base shear:</p> <p><math>1.3 \times K_{eff} \times \Delta_{ELFM}</math></p> <p>Max stresses:</p> <p>Within code limits for short-term loading</p>

### 2.1.5 Design review process

The review process is standardized in Japan. Route 2 (normal constructions) is verified as per any seismic-resisting building by local government. Route 3 (special constructions) is reviewed by a government mandated committee, which may be even run by a private company. This results in reduced time, and it takes in general one or two months (Becker et al., 2010). The permission of MOLIT usually takes more than two months. So the Route 3 will take 3-4 months more time than Route 2.

### 2.1.6 Prototype and control testing

In Japan each device manufacturing company has a catalogue of pre-approved devices. The devices go through an accreditation process run by the government mandated committee following Notification No.1446. Manufactures submit the data required for accreditation. All the isolation devices going into

the building (with the exception of rail sliders) are subjected to a 100% quality control tests. (Becker et al., 2010).

## 2.2 US code (ASCE 7-10)

### 2.2.1 Design spectrum

The current ASCE 7-10 code implements a 2-level design approach in which the structural design forces are designed to the DE event and the isolators and all structural elements below the isolators are designed to the Maximum Considered Earthquake (MCE). The DE event is defined as two-thirds of the MCE event (resulting in MCE being equal to 1.5 times the DE event). The MCE event is based off a risk targeted ground motions. Loosely the MCE event is either an event with a 2% probability of exceedance in 50% years (return period of approximately 2500 years) or for near source locations it is defined by the 1.8 times the deterministic median design event spectra. The design spectrum is based on the SRSS combinations of the horizontal components (or maximum rotated component, NEHRP, 2011). An IF of 1.0 is adopted for all base isolated buildings, regardless of their use. The reference shape of the acceleration design spectrum is represented in Figure 6. It is worth mentioning that in the US, the corner period ( $T_D$  in Figure 6), which defines the transition to the constant displacement range is very large - 8s-12s in California (according to the US hazard map) and hence the displacement demand, defined as pseudo displacement spectra (no specific design displacement spectra is specified in the code), increases even at longer periods, in a similar fashion to what is observed in Figure 4 for the Japanese code.

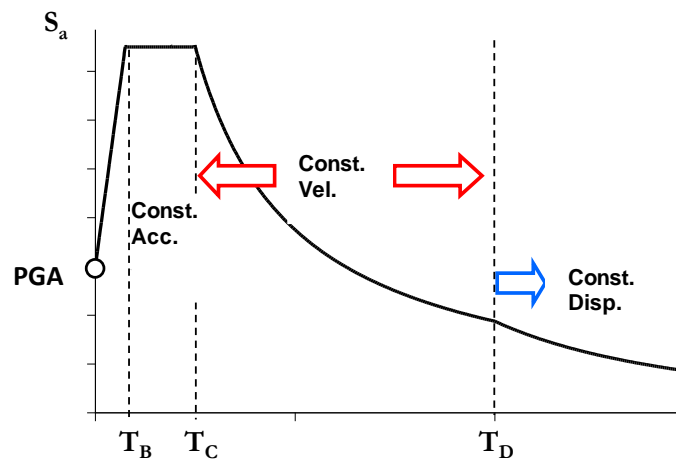


Figure 6. US code DE 5%-damped spectra: reference shape

For NLTHA the code requires the adoption of seven pairs of records scaled to the design spectrum so that the average of the SSRS spectrum of component pairs is not lower than the design value over the period range ( $0.2 T_{1,DE} - 1.25 T_{1,MCE}$ ), with  $T_1$  equal to the effective period ( $T_{eff}$ ) (at both the Design level Earthquake (DE) and the Maximum Considered Earthquake (MCE)) for a base isolated structure (evaluated independently at the two seismic intensity levels), as summarised in Table 7. The average of the 7 TH's are used for design. The design vertical acceleration for base isolated buildings is defined as shown in Equation 3, where  $S_{MS}$  is the short period design acceleration for the MCE event, and  $D$  is the effect associated with gravity loads.

$$E_v = 0.2 \times S_{MS} \times D \quad (3)$$

**Table 7. Earthquake records scaling criteria – US code**

Criteria	Note	General Remarks
$SRSS = Average_{7-pairs} = \left\{ \sqrt{S_{ix}^2 + S_{iy}^2} \right\}_{0.2T_{1,DE}}^{1.25T_{1,MCE}} \geq S_d$	Calculate SRSS spectrum for each pair, and average amongst 7-pairs.	The resultant of the two components is compared against the design value.
<p><math>S_{ix}</math>: spectra for the x-component of record pair i  <math>S_{iy}</math>: spectra for the y-component of record pair i  <math>S_d</math>: design spectrum</p>		

### 2.2.2 Design method

ASCE 7-10, allows the adoption of an equivalent linear analysis, or ELM (the procedure is summarised in section 3, Table 16), a Response Spectrum Analysis (RSA) or NLTHAs. Limitations on the adoption of the ELM apply both on the structural layout and seismicity of the site (Table 8), as well as on the isolator properties (Figure 8 (b)).

**Table 8. Requirements for the application of the ELM – US code**

Parameter	Requirement	Comments
Maximum effective period	3.0s	To allow for an equivalent linear model to be used
Axial force on isolators	No tensile forces	
Soil conditions	Site Class A, B, C, or D	Not allowed for soft clay sites
Seismic intensity level	$S_d(1s) < 0.60g$	Not allowed in high seismicity areas
Superstructure	Regular Max 4 stories or 19.8m height	
Isolators	Equivalent Linear modelling (Figure 8 (b))	

### 2.2.3 Structural modelling

In linear analysis procedures the behaviour of the isolation system is represented by means of equivalent properties (stiffness and damping). The response spectrum reduction factor is defined in the form of tabulated values corresponding to different levels of the equivalent damping  $s$ , as summarised in Table 9 and Figure 8 (a). A full 3D model is used in RSAs, with full representation of bi-directional loading and torsional response. The same applies to NLTHAs. Requirements apply for the adoption of a linear model of the isolators (Figure 8 (b)) and variations in properties of the isolation system should be evaluated by the engineer, based on long term environmental effects and prototype test results, as well as lower and upper bound design scenarios.

**Table 9. Equivalent damping reduction factors for the application of the ELFM – US code**

<b>Equivalent effective damping ratio (%)</b>	<b>Response spectrum reduction factor</b>
$\leq 2$	1.25
5	1.00
10	0.84
20	0.67
30	0.59
40	0.53
$\geq 50$	0.50

#### *2.2.4 Structural performance requirements*

Whilst the capacity of the isolators is defined on the basis of the MCE (2% in 50 years) input, the superstructure is designed with reference to the DE (10% in 50 years) demand. The design forces are based on upper and lower bound properties, the first one evaluated using stiffness properties (secant stiffness of the isolation system), the second one based on the DE displacements (at the centre of rigidity of the isolated layer). The displacement capacity of the isolators is defined by the MCE demand with conservative estimates of their stiffness and damping. The superstructure is designed for a base shear value derived using the maximum expected stiffness using the upper bound properties of the isolators coupled with the maximum displacement demand on the isolators (using their lower bound properties). The elastic base shear so determined can be reduced by a force reduction factor (R) which depends on the structural system adopted and it shall not be greater than 2.0 for all the analysis methods.

As mentioned no IF is used in the code to distinguish between a residential or commercial building and a hospital for seismic loading in base isolated buildings. In principle this would mean that a reduction factor of 2 is allowed for the design of the superstructure, even for a hospital building.

#### *2.2.5 Design review process*

The peer review process is mandatory for isolation projects and is generally managed through the local authorities, who often nominate an external team of peer reviewing engineers.

#### *2.2.6 Prototype and control testing*

In the US, prototype tests and approval are required specifically for every single project unless similarity with past test data can be demonstrated. The designer defines the testing procedure (as per code requirements) and the cost is covered by the client. Testing includes prototype testing on two units and quality control testing on all the isolators installed. Prototype tests are required unless similarity can be demonstrated with other units already approved in the past. The peer review team reviews the prototype test report and sometimes attend the tests.

**Table 10. Structural performance requirements for the application of the ELFM – US code**

Limit state	DE	MCE
Isolators (IS)	-	Design displacement: $\Delta_u < \Delta_{MCE,max}$ where: $\Delta_u$ = displacement capacity $\Delta_{MCE,max}$ = displacement demand [calculated considering lower bound isolators properties and torsional effects]
Superstructure (BLD)	Limited - ductility: Strength reduction factor: max 2 Storey drift: $\theta < 1.5\%$ Design base shear: From upper bound isolators stiffness at the DE level ( $K_{eff,DEmax}$ ) and peak displacement demand (developed with lower bound isolators properties, $\Delta_{ELFM,DEmax}$ ): $V_{base} = 1.0 \times K_{eff,DEmax} \times \Delta_{ELFM,DEmax} / R$ $R = 3/8 R_{BF} \leq 2.0$ R: force reduction factor for the base-isolated building $R_{BF}$ : force reduction factor for a fixed-based structure	-

### 2.2.7 Major Changes Proposed for the ASCE 7-16 Code

Chapter 17 of the present ASCE 7-10 is in the process of adopting some very significant changes for the first time since its original adoption in 1991. The main changes that will be incorporated include the following:

- Modified calculation procedure for the elastic design base shear forces from the DBE event to the MCE event using a consistent set of upper and lower bound stiffness properties and displacements. This modification simplifies the design and analysis process by focusing only on the MCE event.
- Relaxed permissible limits and criteria for the use of the equivalent lateral force (ELF) procedure. This modification minimizes the need to perform complex and computationally expensive non-linear time history analyses to design the superstructure and isolation system on many base isolated structures.
- Use of a project site response spectra easily obtained from the USGS website based on site location rather than requiring a site-specific seismic hazard study. The new USGS web applications provides a multi-period spectra rather than spectral values at two periods (0.2 and 1.0 seconds), previously available.
- Use of nominal properties in the design process of typical isolation bearings specified by the manufacturers based on prior prototype testing.
- These nominal properties are adjusted using the newly incorporated AASHTO 1999 lambda factors to account for response uncertainties and obtain upper and lower bound properties of

the isolation system for the design process.

- New method for the vertical distribution of lateral forces associated with the ELF method of design. This revision incorporates a more accurate distribution of shear over height considering the period of the superstructure and the effective damping of the isolation system and it does not distribute the mass of the isolated base slab vertically as the current provisions do.
- Simplified approach for incorporating a 5% accidental mass eccentricity in non-linear time history analyses.
- Reduction in the required number of peer reviewers on a seismic isolation project from the current panel of 3-5 working as a team to a minimum of one peer reviewer. Also, peer reviewers are not required to attend the prototype tests.
- Calculation procedure to estimate permanent residual displacements that may occur in seismic isolation applications with relatively long period high yield/friction levels, and small yield displacements under a wide range of earthquake intensity.

## 2.3 Europe (EC8)

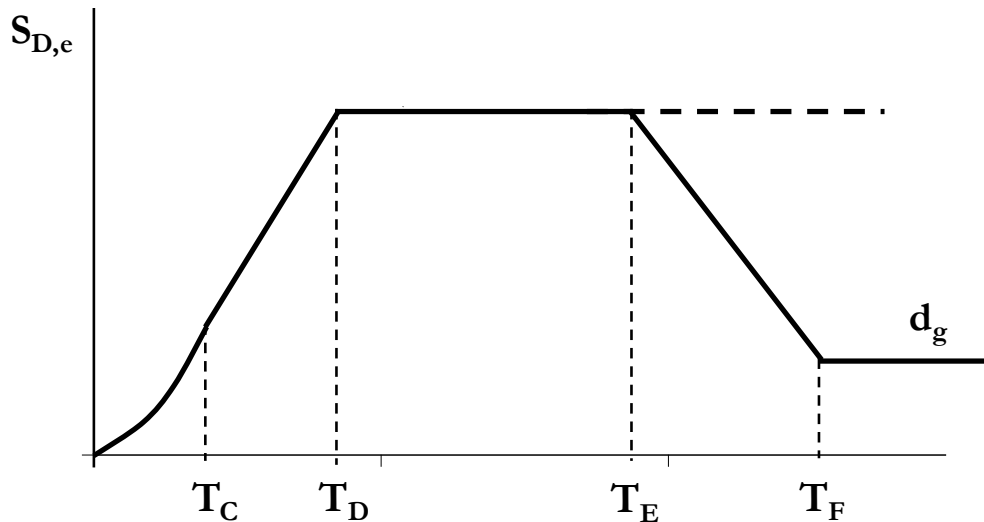
### 2.3.1 Design spectrum

The design spectrum in EC8 is representative of a 500 years return period event, and it is defined with reference to the dominant earthquake component. Both the horizontal components are assumed to be applied simultaneously and in conjunction with the vertical ground acceleration. The code presents a specific design displacement spectrum, not simply derived from pseudo-displacement considerations, which features a corner period at 2s (i.e. the period when the constant displacement regime starts in the acceleration design spectrum) and starts decaying from  $T_E$  (Figure 7) to the level of peak ground displacement for longer periods ( $T_F$  onwards). Hence, even if the design spectral shape is similar to what shown in Figure 6 for ASCE 7-10, the constant displacement range starts much earlier (2s) and the displacement spectrum assumes the reference shape depicted in Figure 7.

Further, the code distinguishes between different importance levels for the structure, by introducing specific IFs, with 1.0 for Importance Class II (ordinary building) to 1.4 for Importance Class IV (critical facilities) (Table 11).

**Table 11. Importance classes and Importance factors (recommended values) – EU code**

Importance class		Importance factor
I	Buildings of minor importance for public safety, e.g. agricultural buildings, etc.	0.8
II	Ordinary buildings, not belonging in the other categories.	1.0
III	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions etc.	1.2
IV	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.	1.4



**Figure 7. EC8 5%-damped spectra: reference displacement shape**

It is worth mentioning that in Italy the most recent design code (NTC08, 2008), building on the lessons learnt from recent major earthquakes in 1997 (Umbria-Marche) and 2002 (Molise), features a more thorough definition of the design earthquake levels, where the hazard intensity is mapped over the country on the basis of the local hazard, and where the design return period does depend on the design life of the structure, its importance level and the targeted limit state, rather than being a priori set to a specific value (say 500 years). In addition, the Italian code includes the definition of a collapse (prevention) limit event (in some way comparable with an MCE event), with varying return period, based on location and building properties as discussed.

**Table 12. Earthquake records scaling criteria – EU code**

Criteria	Note	General Remarks
$Average(S_{i0}) \geq PGA$	Min PGA to control low period ordinates	3 to 6 record pairs: design for peak demands
$Min(S_i)_{0.2T_1}^{2.0T_1} \geq 90\% S_d$	Min scatter over the period range for scaling	7 or more pairs: design for average values
<p><math>S_{i0}</math>: spectral acceleration at 0s for the each record i            PGA: Design peak ground acceleration  <math>T_1</math>: Effective period  <math>S_i</math>: spectral acceleration for the each record i  <math>S_d</math>: design spectral ordinate</p>		

For NLTHA the code requires the adoption of pairs of records (minimum 3, and 7 are required to design for average outcomes) scaled to the design spectrum (Table 12) so as the average value at  $T = 0s$  is not lower than the design ground acceleration and that the minimum value of the spectrum amongst all the considered records is at least equal to 90% of the design spectrum acceleration over the period range  $(0.2 T_1 - 2.0T_1)$ , with  $T_1$  equal to  $T_{eff}$  for a base isolated structure. In 3D models, records are applied simultaneously, following the same combination rule adopted for a RSA (i.e. 100% principal direction + 30% orthogonal component).

### 2.3.2 Design method

The EC8, similarly to the ASCE 7-10, allows the adoption of an equivalent linear analysis, or ELM (the process is summarised in section 3, Table 16), a RSA or NLTHAs. Limitations on the adoption of the ELM apply both on the structural layout as well as on the isolator properties (Table 13).

**Table 13. Requirements for the application of the ELM – EU code**

Parameter	Requirement	Comments
Maximum effective period	3.0s	To allow for an equivalent linear model to be used
Minimum effective period	3.0 T <sub>BLD</sub>	
Seismic intensity level	distance from to the nearest potentially active fault with a magnitude M <sub>s</sub> ≥ 6.5 is greater than 15 km	Not allowed in high seismicity areas
Superstructure	Max size in plan: 50m Regular structure No rocking rotation at the base of the substructure	
Isolators	Linear modelling (Figure 8 (b)) Isolators located above elements supporting gravity loads Vertical stiffness ≥ 150 K <sub>eff</sub> Vertical period not longer than 0.1s	
T <sub>BLD</sub> : period of the fixed based structure K <sub>eff</sub> : effective stiffness of the isolation system		

### 2.3.3 Structural modelling

In linear analysis procedures the behaviour of the isolation system is represented by means of equivalent properties (stiffness and damping). The damping reduction factor is defined in Equation 4, where  $\xi_{eff}$  is the effective damping ratio in %. A full 3D model is used in RSAs, with full representation of bi-directional loading and torsional response. The same applies to NLTHAs. Requirements apply for the adoption of a linear model of the isolators (Figure 8 (b)). Variations in properties of the isolation system, for normal-class buildings (Class I-II, IF ≤ 1.0), provided that extreme (maximum or minimum) values do not differ by more than 15% from the mean values, are neglected.

$$F_h = \frac{10}{\left(5 + \xi_{eff}\right)} \geq 0.55 \quad (4)$$

### 2.3.4 Structural performance requirements

The design of both the structure and the isolators is based on the demand obtained for the DE level. The displacement capacity of the isolators is defined by the actual demand value derived from the analyses amplified by a factor of 1.2 for increased reliability. This, in fact, as mentioned previously with reference to the Japanese code, might be considered as “in lieu” of the collapse limit or MCE-type event. The design displacement should include the contribution associated with long-term deformations and 50% of the design thermal-induced displacements. The superstructure is designed



for the base shear value obtained from the analyses, allowing for the adoption of a maximum force reduction factor (R or behaviour factor q in Europe) of 1.5. Drift limits are set to 0.5%, 0.75% and 1.0%, for buildings with brittle or ductile non-structural elements, respectively (or, in the latter case, if the non-structural elements do not interfere with the seismic response of the main structural skeleton). Provided these drift limits are respected, the superstructure can be designed with no seismic detailing, hence referring to the design code for non-seismic loading (Eurocode 2), without Capacity Design requirements nor detailing requirements for ductility. Design performance criteria for ELMF designs are summarised in Table 14. Once again these are to be taken as minimum by code (law) and should not necessary be considered as typical target design criteria in design practice, specifically when higher importance Classes of buildings are considered.

In fact, it is worth mentioning that the current (and more recent than the Eurocode) Italian code (NTC08) does set more detailed and stringent criteria for designing the isolators, and the structure as well. The NTC08 requires the isolators to be designed for the collapse limit state (rather than the DE level), a minimum design acceleration for the superstructure (0.07g) and detailing of structural members for providing minimum ductility capacity.

**Table 14. Structural performance requirements for the application of the ELMF – EU code**

Limit state	DE
Isolators (IS)	Design displacement: $\Delta_u < 1.2 \Delta_{ELFM} + 50\% \Delta_{DT}$ where: $\Delta_u$ = displacement capacity $\Delta_{ELFM}$ = displacement demand [calculated considering lower bound isolators properties and torsional effects] $\Delta_{DT}$ = thermal induced deformations
Superstructure (BLD)	Limited - ductile: Strength reduction factor (R): max 1.5 $\theta < 0.5\%$ for buildings having brittle non-structural components $\theta < 0.75\%$ for buildings having ductile non-structural components $\theta < 1.0\%$ for buildings having non-structural components not interfering with structural deformations Design base shear: $1.0 \times K_{eff} \times \Delta_{ELFM}/R$

### 2.3.5 Design review process

The reviewing process varies in different countries, however it is generally managed through the local authorities, who may complete the review internally or defer that to an external board.

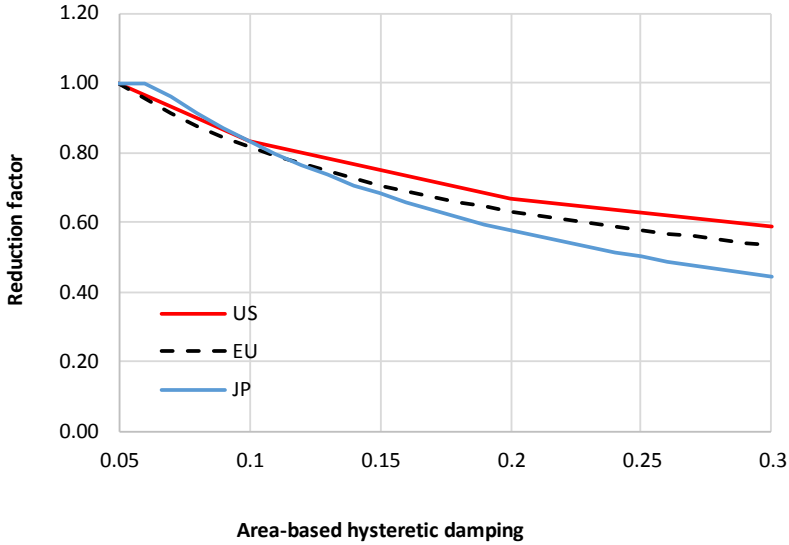
### 2.3.6 Prototype and control testing

The design of seismic isolated structures in Europe does require firstly that the type of units produced by the intended supplier (hence with generic size and properties) has already acquired a technical approval from the government. Furthermore, the European code requires quality control tests to be performed on at least 20% of the total number of devices installed (minimum 4). The latter are based on quasi-static procedures and they might be performed internally by the manufacturer itself. Prototype tests, performed including dynamic procedures, are required on at least two units.

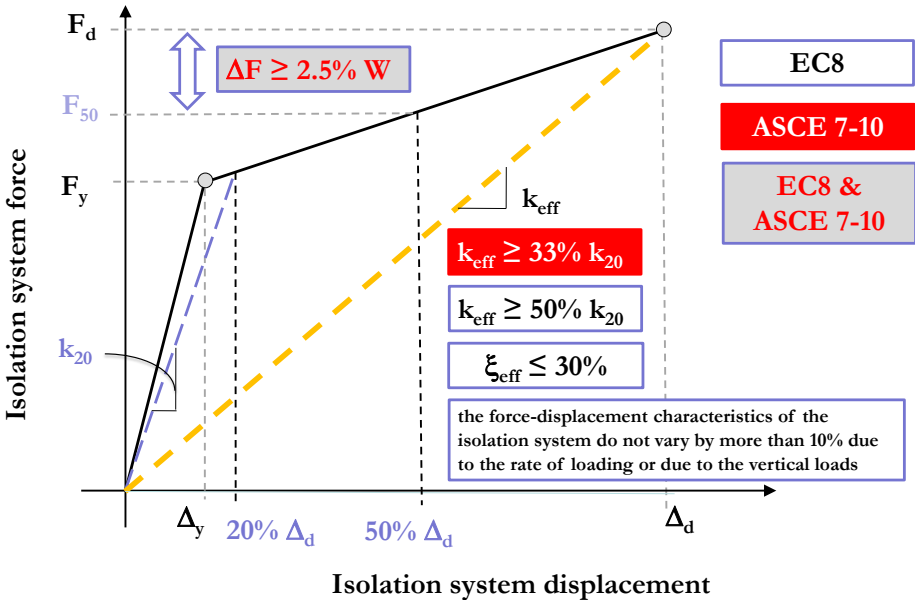
## 2.4 Summary

From the analysis of the three international codes considered (a summary is provided in Table 16), it is noted that, in general, the Japanese regulation framework implements a streamlined process, with approval and reviewing procedures already defined by the government. In Japan a simpler design pattern is in principle available, for small constructions, although, in fact, the latter did not find any

relevant use in practical applications. From a design perspective, the approach in the Japanese code is in general characterized by the implementation of amplification factors on demands and safety factors on capacities, in order to reduce the computational effort (for example when considering torsional effects), to introduce a safety margin on isolator's design towards a larger-than-design event (i.e. collapse limit or MCE-type earthquake), or a protection factor on the design forces of the superstructure (to consider variations in the isolators' properties), when an equivalent lateral force method is implemented and nominal devices properties are used. These coefficients might feature a higher level of conservatism, as for example for the equivalent damping reduction factor (Figure 8, left), however, as it can be observed from the case study presented in the following section, the overall design may not be so more conservative when compared with other codes.



(a)



(b)

Figure 8. Comparison between spectral damping reduction factors in US, European and Japanese codes (a), and requirements for the adoption of a linear model for the isolators in ASCE 7-10 and EC8 (b)

The US code, by virtue of the MCE level adopted for designing the isolators, results in higher demands on these units, while, on the other side, the code allows for a certain level of ductile behaviour of the superstructure, hence with lower design forces, however requiring adequate structural detailing for seismic loading. This distinguishes the US from the Japanese and the European codes, where the design mainly focuses on DE levels and requires the superstructure to respond elastically. Both the EU and Japanese codes also provide an amplification factor of 1.2 on the isolator displacement demand, somehow building some redundancy (on the isolators) for a higher than DE level earthquake.

More so, the Japanese code, introduces a unique amplification factor of 1.3 to the base shear, on the basis of a capacity design approach, when forces are obtained from nominal isolators' properties.

It is worth noting that the EC8 does not represent the most advanced European regulation. The recent Italian Code (NTC08), for example, has been developed further and it introduces a collapse limit, or MCE-type event, for designing the isolators, in parallel to a DE level for designing of the superstructure. However, as ASCE 7-10 does, it includes stringent requirements for the adoption of a linear procedure, hence limiting the selection of the isolators. Of major relevance are also the differences in terms of definition of the equivalent lateral load pattern for designing the superstructure (proportional to the storey mass only – not the height – in EC8), and the introduction of a structural importance level coefficient ( $IF \geq 1.0$ ).

In general, it might be concluded that, with reference to normal or ordinary buildings (Importance Factor,  $IF = 1$ ), ASCE7-10 seems to provide a more conservative design of the isolators (in considerations of the MCE displacement demand). The design forces compared in the case study below are quite similar for the Euro and US codes. The US code requires the same ductile structural detailing as a conventional structure. The EC8 and Japanese code seems to allow less non-ductile structures to be used in conjunction with an isolation system.

Such conclusions change for a structure with an importance factor greater than 1.0. Whilst both Japanese and European regulations see a significant increase in the key design parameters, an ASCE 7-10-based design does not have an importance factor. Thus the displacement demands are the same for all building types and California hospitals are limited to an R-Factor of 1.5. The design criteria for the isolators is also very different between the European and US codes on one side and the Japanese code on the other side, with the latter allowing for much flexible systems, by virtue of setting a minimum level for the tangent (isolated) period and by removing requirements on linear modelling of the isolation system. This obviously may have implications on the reliability of the linear analysis outcomes and on the control of the deformation demand during severe events.

Finally, it is worth recalling that the US and Japanese codes both, in practice, do not include a corner period in the design spectrum (very large - 8s-12s in California - according to the US hazard map). Other codes, such as EC8 or NZS 1170.5, implement a corner period, and hence a constant displacement demand region. In general the definition of a constant displacement plateau seems to depend on the local characteristics of the seismic hazard and it should be thus better calibrated on the basis of a seismic hazard study for each specific region, and neglected if justified by the nature of the ground acceleration records.

**Table 15. Comparison between Japanese, US and European code requirements**

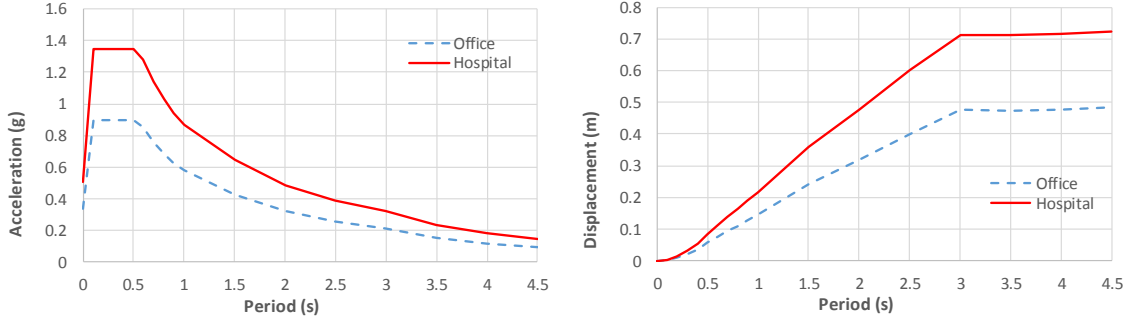
	<b>JP (BSLEO)</b>	<b>US (ASCE 7-10)</b>	<b>EU (EC8)</b>
Design methods	No Calc. / ELM / NLTHA	ELFM / RSA / NLTHA	ELFM / RSA / NLTHA
Design Spectra (IS/BLD)	DE/DE	MCE/DE	DE/DE
Spectrum	Dominant component	Maximum rotated component	Dominant component
Importance factor	Yes	No	Yes
Vertical load	Included	Included	Included
Ageing/dispersion	1.2	From tests	Use average isolators' properties for IL1-IL2 structures, if variations are within $\pm 15\%$
Safety factor on isolation capacity	elastic ID = 0.8 – sliding/friction = 0.9	Implicit in the MCE design level	1.2
Torsion in ELM	1.1 Max ecc. 3%L (+ 0% accidental ecc.)	Calculated No limit (+ 5% accidental ecc.)	Calculated Max ecc. 2.5%L (+ 5% accidental ecc.)
Clearance	>> Max Design disp.	Max Design disp.	Max Design disp.
BLD requirements	Elastic $V_{base} = 1.3 V_{ELM}$ CD and ductility req.: NO	low ductile (max R = 2) $V_{base} = 1.0 V_{ELM,DE}$ CD and ductility req.: YES	low ductile (max R = 1.5) $V_{base} = 1.0 V_{ELM}$ CD and ductility req.: NO
Modelling	Simple 2D, even for NLTHA	2D for ELM, otherwise 3D	2D for ELM, otherwise 3D
ELM basic conditions	$T2 > 2.5s$ $Fy > 0.03W$	More stringent requirements on the implementation of a linear model may not allow for large T2	More stringent requirements on the implementation of a linear model may not allow for large T2
Design review	Standard government procedure	Mandated by the code	Mandated by the code
Testing	Catalogue already approved by government	Engineer to define	Engineer to define

### 3 CASE STUDY

In order to provide a more clear appreciation of the ramifications of the design criteria and approaches between the three major international codes presented above a Case Study Building (CS) is used as a design example.

The building is a virtual Moment Resisting Frame (MRF) building (material non-specific, either

concrete, steel, timber or combination of the above) featuring three storeys and a total seismic weight (including the basement level) of 13,718 kN (Figure 10). The design level (DE) spectrum is assumed equal to that defined in NZS 1170.5 for an IL2 structure (assume an Office building (O)), considering soil class D and a hazard factor of  $Z=0.3$ . The vertical acceleration component is neglected for the purpose of the comparison presented in this paper. The assumed building geometry, and properties of the isolators (in ASCE and Eurocode designs), satisfy the requirements for the ELM to be adopted. The ELM design equations and the corresponding data for all codes are summarised in Table 16. The design is performed assuming two alternative device solutions, namely Lead Rubber Bearings (LRB, alone or combined with low-friction flat sliders), or Friction Pendulum Sliders (FS) only.



**Figure 9. Design (5%-damped) acceleration response spectrum (left), and pseudo-displacement spectrum (right)**



**Sample building:**  
 Moment resisting frame building: 4 storeys  
 Interstorey height = 3.5m  
 4 x 2 bays: 8.5m span  
 Seismic weight: 13,718 kN  
 (Basement: 35%, 1<sup>st</sup>: 28%, 2<sup>nd</sup>: 28%, Roof: 9%)

**Figure 10. Case Study building: summary of geometric layout**

The design is performed with reference to an isolated period of 2.7s and a yield level of the isolation system of 5% and 10% of the total supported seismic weight for the LRB-design and the FPS-design, respectively, in order to compare the outcomes of the designs from different codes against a fixed parameter. Consequently, the resulting designs might not represent the optimal solution for each particular set of devices, nor reflect the construction practice of each country. For such purpose some key design features of the isolators will be realistic but not optimized, depending on the isolator type, as it would occur in real design practice. Preliminary design values for the isolators are provided in Table 18, where the size and number of isolator units adopted is selected so as to develop a global response consistent with the design outcomes obtained from the ELM (Table 16 and Table 17). It is worth recalling that, for the LRB, the diameter of the lead plug will dictate the yield threshold, the height of the rubber will govern the post-elastic stiffness and the diameter will govern the displacement capacity; for the FS, the friction coefficient sets the “equivalent” yield threshold, while the radius of curvature completely defines the isolated period. A combination of friction flat sliders and LRBs is used to develop systems that have adequate load capacities at larger displacement demands. Upper and lower bound properties of the isolators are considered in the calculations performed for the EC8 and ASCE7-10, as required in these codes, assuming an amplification or upper bound factor of 1.25 on the second slope stiffness and yield/friction level and a reduction or lower bound factor of 0.8 on the second slope stiffness and yield/friction level.

**Table 16. Case study: ELFM procedure: LRB design**

<b>Parameter</b>	<b>JP (BSLEO)</b>	<b>US (ASCE 7-10)</b>	<b>EU (EC8)</b>
	<b>Case Study Office / Hospital</b>	<b>Case Study Office / Hospital</b>	<b>Case Study Office / Hospital</b>
Tangent period, $T_2$ (s)	2.7 / 2.7	2.7 / 2.7	2.7 / 2.7
Isolator Yield force ratio ( $F_y/W$ )	0.05 / 0.05	0.05 / 0.05	0.05 / 0.05
Centre of mass displacement, $\Delta_{CM}$ (mm)	226 / 506	582 / 582	318 / 526
Factor for torsion (TF)	1.1 / 1.1	1.24 / 1.24	1.15 / 1.15
Safety factor on device capacity, $\beta$	0.8 / 0.8	1.0 / 1.0	1.2 / 1.2
Max design displacement, $\Delta_d$ (mm)	426 / 706	720 / 720	438 / 725
Total elastic base shear ( $V_{b,e}/W$ )	0.18 / 0.33	0.31 / 0.31	0.18 / 0.32
Total design base shear ( $V_b/W$ )	0.23 / 0.43	0.16 / 0.21	0.12 / 0.21

Furthermore, the outcomes, in terms of design base shear for the superstructure are based on the assumption that the maximum force reduction factor (or ductility factor or reduction factor) allowed in each respective code is used. For instance, the design for the Hospital-type building (H) according to the ASCE 7-10 is performed under the assumption that a maximum R Factor of 1.5 is required by the local authorities (OSHPD) in California (the same assumption is adopted here also for the EC8 design). The design is finally repeated assuming the same building will feature a higher importance class (for example a Hospital, H), assuming an IF of 1.4 in European codes. The design spectra for the “Office-type” building scenario and the “Hospital-type” scenario are represented in Figure 9. A summary of the major properties of the designed isolators are provided in Table 9. Note: a) the ASCE 7-10 design accounts for an MCE type event (here assumed as 1.5 the reference IL2 design spectrum) and b) will not change with respect to the office building; c) the allowance for the adoption of more “flexible” systems in the JP design (because of no limitations on linear modelling of the devices).

From the analysis of the results obtained for this simple case study (Table 16, Table 17 and Table 18), it is observed that the European and US codes have similar levels of design forces for the structure and the only major difference is the lower displacement demand required for an office type building in the Eurocode. Consideration of a MCE level, linear modelling requirements on the isolators and the introduction of an importance factor for the building are the primary sources of difference. The shape and the definition of the design spectra also play an important role if these codes are applied in other countries.

**Table 17. Case study: ELM procedure: FS design**

<b>Parameter</b>	<b>JP (BSLEO)</b>	<b>US (ASCE 7-10)</b>	<b>EU (EC8)</b>
	<b>Case Study Office / Hospital</b>	<b>Case Study Office / Hospital</b>	<b>Case Study Office / Hospital</b>
Tangent period, $T_2$ (s)	2.7 / 2.7	2.7 / 2.7	2.7 / 2.7
Isolator Yield force ratio ( $F_y/W$ )	0.10 / 0.10	0.10 / 0.10	0.10 / 0.10
Centre of mass displacement, $\Delta_{CM}$ (mm)	105 / 238	402 / 402	213 / 362
Factor for torsion (TF)	1.1 / 1.1	1.24 / 1.24	1.15 / 1.15
Safety factor on device capacity, $\beta$	0.9 / 0.9	1.0 / 1.0	1.2 / 1.2
Max design displacement, $\Delta_d$ (mm)	305 / 438	500 / 500	293 / 500
Total elastic base shear ( $V_{b,e}/W$ )	0.16 / 0.23	0.31 / 0.31	0.23 / 0.31
Total design base shear ( $V_b/W$ )	0.20 / 0.30	0.16 / 0.21	0.16 / 0.21

**Table 18. Case study: reference design parameters for the seismic isolation units**

<b>Displacement Demand</b>	<b>LRB's - Sliders</b>	<b>LRB Diameter</b>	<b>Rubber Height</b>	<b>Total Height</b>
300 mm	15 - 0	650	280	460
440 mm	10 - 5	700	210	360
500 mm	8 - 7	700	170	305
720 mm	8 - 7	950	280	510

**FS:** friction coefficient = 10% - Radius of curvature = 1.8m (the displacement demand affects the size of the bearings only).

Consequently, the application of overseas codes for the design of base isolated buildings in New Zealand, as required in practice in absence of a dedicated standard (or section), will inevitably lead to different designs and hence building performances, with consequent lack of homogeneity between structures of the same class. Furthermore, the consistency with the other NZ standards cannot be guaranteed, particularly in terms of design hazard levels and elastic spectra (definition and shape of the spectrum, scaling of records) as well as design/damped spectra (ductility factor and structural performance factor  $S_p$ ) soil classes and requirements for specific local hazard and site-specific studies, and requirements on the structural performance.

In summary, in order to ensure a reliable and code-compliant implementation of base isolation design in New Zealand, the following aspects should require particular attention and a dedicated discussion:

1. Design spectral shape: NZS 1170.5(2004) implements a constant displacement region for periods longer than 3s, which is potentially un-conservative for the design of base isolated

systems with longer periods in the 4 to 6 second range, if not otherwise justified by the local nature of the seismic motion. For example, recent research on Probabilistic Seismic Hazard Study (PSHA) to define the long period seismic hazard for the Italian territory (Faccioli and Villani, 2009), produced new hazard maps for the country in terms of horizontal displacement response spectral ordinates for a wide range of vibration periods, from 0.05s up to 20s, showing the tendency for the displacement demand to remain constant, or decrease, at long periods.

2. The appropriate value if not, in more general sense, the actual role of  $S_p$  factor as included in NZS1170.5 when dealing with base isolated structures: careful consideration should be given in particular when considering the evaluation of the design displacement (demand) on the isolators. While all the international codes reviewed do adopt reduction factors on the elastic base shear for the design of the superstructure, none seems to incorporate a reduction factor similar in concept or in practice to the  $S_p$  factor of the NZ standard, to the design displacement (demand). If any, there are actually magnification factors on the design displacement of the isolators.
3. Criteria requirements for a local seismic hazard study to be performed (under which circumstances and depending on the design approach).
4. Design levels (DBE and/or MCE?), importance factors and additional safety/magnification factors that should be used – these may differ for IL2 and IL4 buildings.
5. Design boundaries on the adoption of simplified ELF procedures.
6. Performance objectives/requirements for the isolators and the structure.
7. Quality Control testing procedures and acceptance requirements for isolators
8. Capacity design process and consideration of variations in the isolators' properties.
9. What level of reduction factors for the superstructure base shear and drift/ductility limits for the superstructure response
10. Structural detailing requirements (fully ductile, nominally ductile, with or without capacity design considerations).
11. Consideration of floor acceleration spectra and requirements for non-structural components.
12. Peer review process.

#### 4 CONCLUSIONS

The overview of three major international codes for the design of base isolated buildings has outlined the current differences, in some cases relevant, in the design/approval process as well as in the structural design requirements and calculation methods. With this in mind, and considering the current lack of a NZ standard on the subject (as well as the lessons learnt from the Canterbury earthquakes sequence and the revamped interest into a wider implementation of cost-efficient low-damage structural systems including base-isolation), it is in the authors' opinion that the engineering community in the country should effectively collaborate at national and international level towards the development of a NZ-specific design "code", or endorsed guidelines, for base-isolated buildings.

Such document could bring together the current best practice in NZ and overseas, whilst assuring consistency with the current standards as well as homogeneity in the design of base-isolated structures throughout the country.

#### REFERENCES

- AASHTO. 1999. Guide Specifications for Seismic Isolation Design, American Association of State Highway and Transportation Officials, Washington (D.C., US).
- ASCE 7-10. 2010. Minimum Design Loads for Buildings and Other Structures, ASCE, Reston (Virginia, US).



- Becker, T.C., Furukawa, S, Mahin, S.A. & Nakashima, M. 2010. Comparison of US and Japanese Codes and Practices for Seismically Isolated Buildings, 2010 Structures Congress, ASCE (US).
- COSMOS. 2010. Practical application of the new ASCE 7-10 required procedures for determining site specific ground motions, COSMOS, Richmond, CA (US), November 2010.
- EN 1998-1:2004. Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings, CEN, Brussels (Belgium, EU).
- Faccioli, E. & Villani, M. 2009. Seismic hazard mapping for Italy in terms of broadband displacement response spectra. *Earthquake Spectra* 25, 515-539.
- Feng, D., Liu, W. Masuda, K., Wang, S. & Huan, S. 2006a. – A comparative study of seismic isolation codes worldwide - part I: Design spectrum, 1<sup>st</sup> ECEES, paper N. 63, Geneva, 2006.
- Feng, D., Liu, W. Masuda, K., Wang, S. & Huan, S. 2006b. – A comparative study of seismic isolation codes worldwide - part II: Response analysis, 1<sup>st</sup> ECEES, paper N. 66, Geneva, 2006.
- Higashino, M. & Okamoto, S. 2006. Response control and seismic isolation of buildings, Taylor & Francis, New York (US).
- Kani, N. & Otani, S. 2002. Japanese State of Practice in Design of Seismically Isolated Buildings, The Fourth U.S.-Japan Workshop on Performance-Based Earthquake Engineering Methodology for Reinforced Concrete Building Structures, 22–24 October 2002, Toba, Japan, PEER 2002/21, DEC. 2002, Pacific Earthquake Engineering research center, Berkeley (California, US).
- Kani, N., Feng, D. & Sera, S. 2010. Performance Based Design of Seismically Isolated Buildings in Japan, Workshop: Bridges seismic isolation and large scale modeling, St.-Petersburg, Russia, June 29th - July 3rd, 2010.
- Midorikawa, M., Okawa, I., Iiba, M. & Teshigawara, M. 2003. Performance-Based Design Code for Buildings in Japan, *Earthquake Engineering and Engineering Seismology*, Vol. 4, No. 1.
- Nakashima, M. & Chusilp, P. 2003. A Partial View of Japanese Post-Kobe Seismic Design and Construction Practices, *Earthquake Engineering and Engineering Seismology*, Vol. 4, No. 1
- NEHRP. 2011. Selecting and scaling earthquake ground motions for performing response-history analyses, NIST GCR 11-917-15 report, November 2011.
- NTC08. 2008. Norme tecniche per le costruzioni– DM 14-01-2008, Italy
- NZS 1170.5:2004. Structural Design Actions – Part 5: Earthquake actions – New Zealand, Standards New Zealand, Wellington (NZ).