Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 48, No. 2, June 2015

DESIGN OF BASE-ISOLATED BUILDINGS: AN OVERVIEW OF INTERNATIONAL CODES

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(Submitted *July 2014*; Reviewed *September 2014*; Accepted *February 2015*)

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/devices with controlled force capacity, significant lateral deformation capacity and (often) enhanced energy dissipation. Such a design solution has shown its effectiveness in protecting both 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 earthquake sequence struck 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 of a 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.

Note: at the time of publishing this paper a NZSEE Working Group has been recently established and work is in progress to develop design guidelines for base-isolated buildings in New Zealand, with the support and endorsement of MBIE (Ministry of Business, Innovation and Employment) and EQC (Earthquake Commission).

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 by Feng *et al.*,[1, 2] 2006a, b), 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 mainly the Japanese (Building Standard Law Enforcement Order, (BSLOEO) 2000), the US (ASCE 7-10) [3] and the European (Eurocode 8, EC8) [4] codes. References to the latest draft of the US code (ASCE7-13) as well as to the latest revision of the Italian code, (NTC08) [5] will also be provided for information on more recent trends. Examples of base isolated buildings in these respective countries are shown in Figures 1-3.

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) for the design of an Office-type building and a Hospital, using friction pendulum sliders or lead rubber bearings.

INTERNATIONAL CODES

Japanese Code (MRIT, 2000) [6]

The regulation framework in Japan is articulated in a Building Standard Law (the last revision was in 1998 and introduced performance-based design concepts) and associated Enforcement Order (Building Standard Law Enforcement Order [MRIT], 2000) [6], 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 [7]. In 2010 it was estimated that 2,500 buildings and 3,500 detached houses were using seismic isolation technology [8].

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 construction" [actually never used in practical applications].

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- Route 2: Notification 2009 route for “normal construction” (ELFM) [estimated to be adopted in only 10% of applications].
- Route 3: Non-Linear Time-History Analyses, (NLTHA) for special construction [the most widely adopted approach, covering an estimated 90% of practical applications].

Design Spectrum

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

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 adopts a SLS earthquake with a return period of 1/25 (25 years). The new return period factor for the serviceability limit state ($R_s = 0.33$) in the Canterbury earthquake region is in fact due to the higher short-term seismicity considerations and not to a revision of the return period for SLS level. A lower return period at SLS could lead to less demanding and thus less conservative requirements to protect damage to non-structural elements and content.

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).

Table 1: Earthquake levels and performance criteria – Japanese code [7].

Intensity level	**L1 (damage limitation) ~ SLS**	**L2 (life safety) ~ DE**
	50	500
Superstructure performance target	R2: Elastic ($\theta < 1/500$)	R2:Elastic limited ($\theta < 1/300$)
Substructure performance target	R3: Elastic ($\theta < 1/300$)	R3: Elastic limited ($\theta < 1/150$)
Isolators performance target	$\gamma < 1 \sim 1.5$	$\gamma < 1.5 \sim 2.5$ $\sigma_{tension} < 0$ (No tension) (Route 3 only: $\sigma_{tension} <$ 1MPa in rubber bearings and tension allowed in rail sliders)

** estimated; θ: Interstorey drift; $\sigma_{tension}$: Tensile strain in isolators; γ: Shear strain in isolators;*

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

$$S_a(T) = ZG_s(T)S_0(T) \qquad (1)$$

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, $V_s > 400$ 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 [9]. Sample design spectral shapes are reported in Figure 4. In engineering practice, the $G_s(T)$ is usually calculated iteratively based on the investigated Shear Wave Velocity's or N-values (from Standard Penetration Tests) and types for the soil profile rather than directly using the coefficients defined in the code in Route 2. The (pseudo-) displacement spectra do not feature a corner period for the transition to a constant displacement demand for long-period structures and therefore there is no artificial "cap" to the displacement demand which increases linearly for long periods.

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 to consider 6 records. The design is based on the maximum response values. For the special case of time history analyses, the Ministry entrusts several Authorities to overview the process.

The Authorities, which may be private company or foundations, have reference guidelines, requiring the adoption of the so called "golden set" of records, 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 0.5 m/s for Level 2 input (~ DE), and 0.25 m/s for Level 1 (**~ SLS)**. In addition to these ground motions, a suite of three regionally specific motions is developed, either using artificially simulated or natural records, scaled (in the frequency domain, [9] to match the design spectrum. Usually, the design set includes the JMA Kobe record [10]. The compatibility criteria with the design spectrum are typically defined as summarised in Table 2, although they are not code requirements but rather common "good practice". For the input ground motion, a time history analysis method using equivalent linear analysis or a non-linear model are usually used to obtain $G_s(T)$. The design spectrum is defined as the dominant component.

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 the adoption of a specific Importance Factor in practical applications. Typical values are IF=1.25 for public buildings such as schools and IF=1.5 for essential facilities, such as hospitals. The 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 with normal class buildings.

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.

Route 2: Notification 2009 route for normal construction. Under this path, the building can be designed on the basis of a 2D linear equivalent static analysis (or ELFM), subject to the approval from a building official and ensuring that the requirements outlined in Table 5 are satisfied. The code defines "normal constructions" as:

- Buildings with total height (H) ≤ 60 m (approximately 15-20 storeys),
- 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 are not allowed).

ELFM allows for the definition of the design displacement (Δ_{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.

Route 3: Alternative 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.

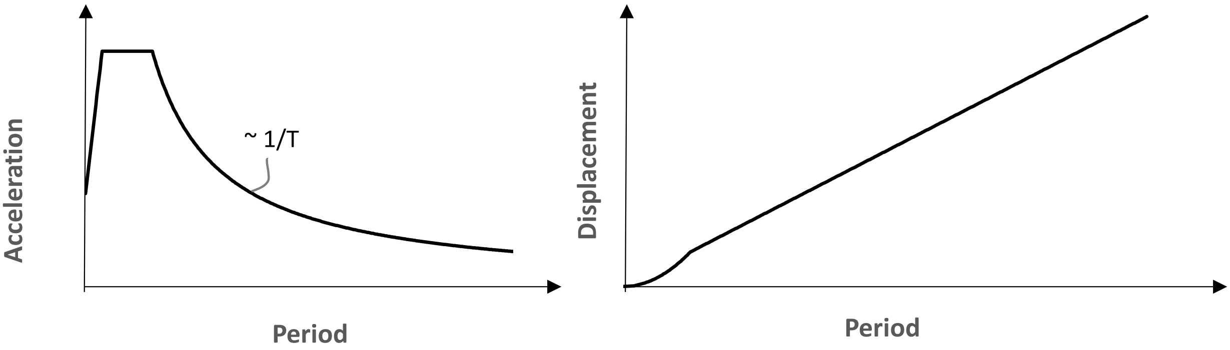


Figure 4: 5%-damped design spectral shape: acceleration (left), displacement ordinates (right) – Japanese code.

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 ≥ 85% target	Scaling is referred to the pseudo velocity ordinates.
$COV \leq 0.05$	Low dispersion	Consideration of the full period range (even if not clearly specified) Compatibility is assessed for each record separately
$\left\|1 - \varepsilon_{ave}\right\| \leq 0.02$	Average value close to target	
$SI_{ratio} = \left(\int_1^5 S_{psv}(T)dT / \int_1^5 DS_{psv}(T)dT \right) \geq 1.0$	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

Table 3: Definition of "small constructions" – Japanese code.

Scenario	Requirement	
1	total floor area	≤ 100 m^2
2	Timber construction	number of storeys ≤ 2 two storeys or total floor area ≤ 500 m^2 and Height ≤ 13 m
3	Buildings, other than timber construction	single-storey or total floor area ≤ 200 m^2

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

Parameter	**Requirement**		**Comments**
[total floor area x storey] / [# isolators]	≤ 15 m^2 per isolator		To provide redundancy and redistribution of vertical loads
[total yield strength of the isolation system (F_y) / [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 favour the adoption of isolator devices 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)] / [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	≥ 350 mm		
Equivalent damping at design displacement	≥ 20%		
Ensure appropriate load transfer from the superstructure	Locate isolators at the base of columns/walls		
Minimum tangent period	2.5 s (2.0 s for building height < 13 m)		Isolated period referred to the secant 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 (centre of) mass stiffness eccentricity	3% of plan dimension (0.03b)		

Table 5: Requirements for the application of the ELFM (Route 2) – Japanese code.

Parameter	**Requirement**	**Comments**
Maximum Building height	≤ 60 m	
Soil conditions	Rock or firm ground	
[total Fy 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	>2.5 s	Isolated period referred to the secant 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)

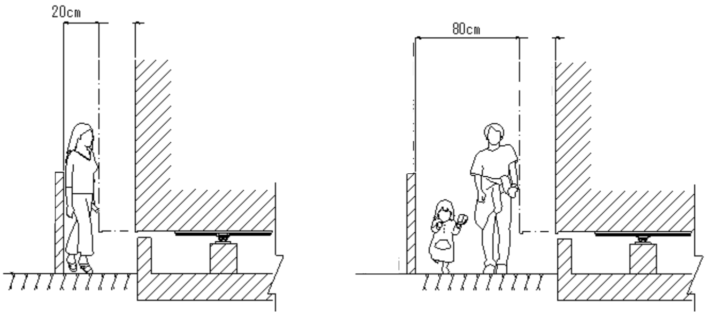


***Figure 5: Required clearance around the perimeter of the building: for maintenance only (left), and when the area is used as general public passage/walkway for non-residents (right) – Japanese code [http://www.menshin.biz/?q=node/351]*.**

Table 6: Structural performance requirements for the application of the ELFM – Japanese code.

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

Structural Modelling

Where the ELFM is used, the contributions of the energy dissipation provided by the isolators is considered through a damping reduction factor R (F_h in the Japanese code) which multiplies the (5% damped) spectral ordinates $S_a(T)$ to obtain the design values (Equation 2, Figure 9 (a)).

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

where ξ_v (h_v in the Japanese code) represent the pure viscous-type behaviour of the isolators/devices (does not cover the structural inherent “viscous” damping), and $\xi_{eff,d}$ (h_d in the Japanese code) represents the area-based equivalent damping coefficient provided by the seismic isolators and hysteretic devices in the seismic isolation layer. A reduction factor is applied to the area-based hysteretic contribution to account for the actual work-energy dissipation contribution during the dynamic response

When NLTHA are used the superstructure is modelled as a non-linear shear-type system, with the shear-type elements usually derived on the basis of the force-displacement response obtained from 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 [9]. The result is thus a very simple equivalent Multi-Degree-of-Freedom column model.

Structural Performance Requirements

With reference to the ELFM (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. A safety factor (β) is applied to the isolator's capacity. This can somehow be considered as "in lieu" of the collapse prevention limit state or Maximum Considered Earthquake (MCE) event as used or implied in other international codes as later discussed. In addition, it is worth noting 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 changes of mechanical characteristics, due to changes to the environmental conditions and decay with time of the devices properties, 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.

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 managed also by a private company. This results in reduced time, and it takes in general one or two months [10]. The permission of MOLIT usually takes more than two months. So the Route 3 will typically take 3-4 months more time than Route 2.

Prototype and Control Testing

In Japan each device manufacturing company has a catalogue of pre-approved devices. The devices go through an accreditation process managed by the government mandated committee following Notification No.1446. The manufacturers submit the data required for accreditation. All the isolation devices used into the building (with the exception of rail sliders) are subjected to quality control tests [10].

US Code (ASCE 7-10)

Design Spectrum

The current ASCE 7-10 code [3] implements a 2-level design approach where 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) event. The DE event is defined as two-thirds of the MCE event. . In simple terms the MCE_R event (following the same notation adopted in the ASCE code) is either an event with a 2% probability of exceedance in 50% years (return period of approximately 2,500 years) or, for near source locations, as 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) [11]. An Importance Factor IF= 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. The US provisions have a corner period (T_D in Figure 6), which defines the transition to the constant displacement range. For California T_D =8-12 s and hence the displacement demand, defined as pseudo displacement spectra, increases even at longer periods, with no artificial cap in a similar fashion to the Japanese code (as shown in Figure 4).

For NLTHA the code requires the adoption of seven pairs of records scaled to the design spectrum so that the average of the SRSS 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 $T_{1,DE}$ and $T_{1,MCE}$ corresponding to the effective periods (T_{eff}), based on the secant stiffness at the design displacement at the Design level Earthquake (DE) and the Maximum Considered Earthquake (MCE_R), respectively for a base isolated structure. These two effective periods are independently evaluated at the two seismic intensity levels, as summarised in Table 7. The average of the 7 TH's response is used for design. The design vertical acceleration for base isolated buildings, E_v, is defined as shown in Equation 3, where $S_{MS}(T)$ is the short period spectral design acceleration for the MCE event.

$$E_v = 0.2 \times S_{MS}(T) \tag{3}$$

Design Method

ASCE 7-10, allows the use of an equivalent linear analysis, or ELFM (the procedure is summarised in section 3, Table 17), a Response Spectrum Analysis (RSA) or NLTHAs. Limitations on the adoption of the ELFM apply to both the structural layout and seismicity of the site (Table 8), as well as to the isolator properties (Figure 9 (b)).

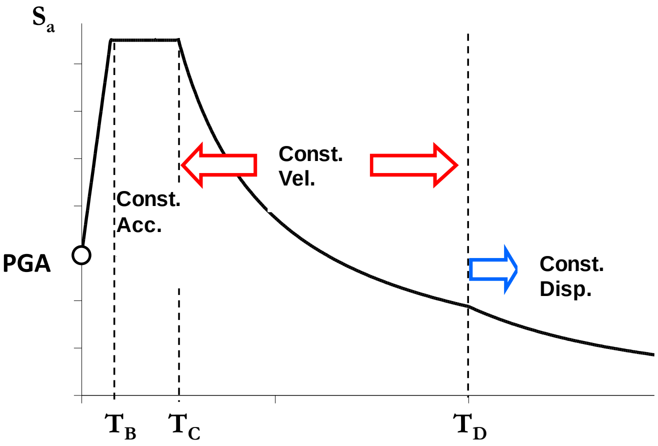


Figure 6: US code DE 5%-damped spectral (acceleration) shape.

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.

S_{ix}: 5% damped acceleration response spectra for the x-component of record pair i

S_{iy}: 5% damped acceleration response spectra for the y-component of record pair i

S_d: 5% damped design acceleration spectrum

Table 8: Requirements for the application of the ELFM – US code.

Parameter	Requirement	Comments
Maximum effective period	$T_{eff} \leq 3.0$ s	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	$S_d(T=1\text{ s}) < 0.60g$	Not allowed in high seismicity areas
Superstructure	- Regular - Max 4 stories or 19.8 m high	
Isolators	Equivalent Linear modelling (Figure 9 (b))	

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

Equivalent effective damping ratio (%)	Response spectrum reduction factor R
≤ 2	1.25
5	1.00
10	0.84
20	0.67
30	0.59
40	0.53
≥ 50	0.50

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, as summarised in Table 9 and Figure 9 (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 when using linear model of the isolators (Figure 9 (b)). Expected 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.

Structural Performance Requirements

Whilst the capacity of the isolators is defined on the basis of the MCE_R (2% in 50 years) input, the superstructure is designed with reference to the DE (10% in 50 years) demand, as summarized in Table 10. The design forces are based on upper and lower bound properties, the first one being 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 from 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_I) which depends on the structural system adopted and can be taken as 3/8 R_{BF} (where R_{FB} is the force reduction factor for a fixed-based structure) but no greater than 2.0 for all the analysis methods.

As mentioned, in the case of base isolated buildings, no IF is used in the code to distinguish between a residential or commercial building and a hospital. In principle this would mean that a reduction factor $R_I = 2$ is allowed for the design of the superstructure, including an essential facility.

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

Component	DE	MCE
Isolators (IS)	-	- Design displacement: $\Delta_u \geq \Delta_{MCE,max}$ where: Δ_u = displacement capacity $\Delta_{MCE,max}$ = displacement demand [calculated considering lower bound isolators properties and torsional effects]
Superstructure (BLD)	- Limited ductile: Force reduction factor (R): 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	

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 review engineers.

Prototype and Quality Control Testing

In the US, prototype tests on two units are required unless similarity with past test data can be demonstrated. Quality control testing, required for every project on all the isolators, includes 3 cycles of combined-compression shear at the design displacement. The peer review team reviews the prototype test report and sometimes attends the tests.

Major Changes Proposed for the ASCE 7-16 Code

Chapter 17 of the present ASCE 7-10 [3] provisions is in the process of adopting some significant changes for the first time since its original adoption in 1991. The main changes that have been approved for the ASCE 7-16 version include the following:

- Modified calculation procedure for the elastic design base shear forces from the DE 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 limits 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 nominal properties of the isolators in the design process specified by the manufacturers, based on prior prototype testing.
- These nominal properties are adjusted using the newly incorporated AASHTO 1999 λ factors to account for response and aging uncertainties to 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.

Europe (EC8)

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 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 $T_D = 2$ s (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 that shown in Figure 6 for ASCE 7-10, the constant displacement range starts earlier (2 s) 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).

For NLTHA the Eurocode 8 [4] requires the adoption of pairs of records (minimum of 3 with design for max response value, or minimum of 7 to design for average outcomes) scaled to the design spectrum (Table 12) so as the average value at T = 0 s 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 [$0.2T_1$ - $2.0T_1$], with T_1 equal to the effective period T_{eff} for a base isolated structure (based on secant stiffness to the target/design displacement of the isolators) . In 3D models, records are applied simultaneously, following the same combination rule adopted for a RSA (i.e. 100% principal direction + 30% orthogonal component).

Design Method

The EC8, similarly to the ASCE 7-10, allows the adoption of an equivalent linear analysis, or ELFM (the process is summarised in section 3, Table 17), a RSA or NLTHAs. Limitations on the adoption of the ELFM apply both to the structural layout as well as to the isolator properties (Table 13).

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

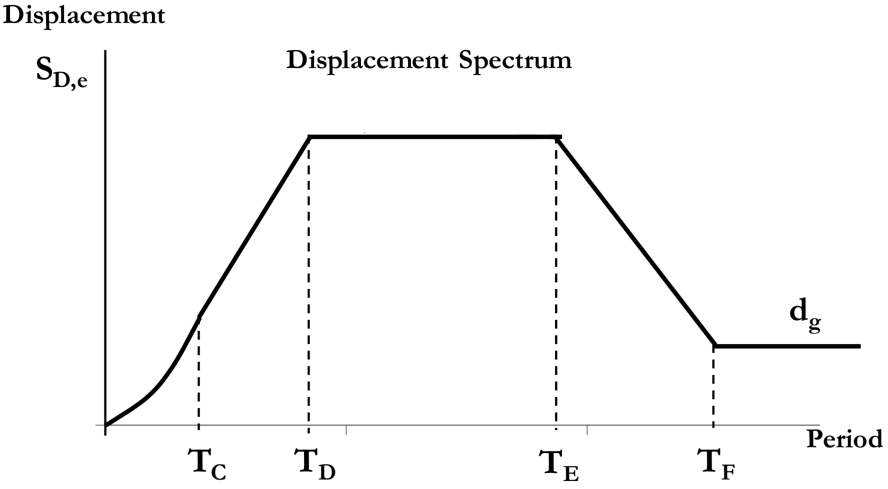


Figure 7: EC8 5%-damped spectra: reference displacement shape.

Table 12: Earthquake record 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

S_{i0}: spectral acceleration at 0s for the each record i

PGA: Design peak ground acceleration

T_1: Effective period at the design displacement

S_i: spectral acceleration for the each record i

S_d: design spectral ordinate

Table 13: Requirements for the application of the ELFM – EU code.

Parameter	Requirement	Comments
Maximum effective period	$T_{eff} \leq 3.0$ s	To allow for an equivalent linear model to be used
Minimum effective period	$T_{eff} \geq 3.0\ T_{BLD}$	
Seismic intensity level	R > 15 km with R= distance from the nearest potentially active fault with a magnitude $M_s \geq 6.5$	Not allowed in high seismicity areas
Superstructure	- Max size in plan: 50 m - Regular structure - No rocking rotation at the base of the substructure	
Isolators	- Linear modelling (Figure 9 (b)) - Isolators located above elements supporting gravity loads - Vertical stiffness $K_v \geq 150\ K_{eff}$ - Vertical period of the isolated structure $T_v \leq 0.1$ s	

T_{BLD}: period of the fixed based structure

K_{eff}: effective stiffness of the isolation system (secant stiffness at the design displacement)

Table 14: Structural performance requirements for the application of the ELFM – EU code.

Component	DE
Isolators (IS)	Design displacement: $\Delta_u \geq 1.2\ \Delta_{ELFM}$ where: Δ_u = displacement capacity Δ_{ELFM} = displacement demand [calculated considering lower bound isolators properties and torsional effects]
Superstructure (BLD)	- Limited ductile: Strength reduction factor (q): 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 x K_{eff} x Δ_{ELFM} / q K_{eff}: effective stiffness of the isolation system (secant stiffness at the design displacement)

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 F_h (η in the EC8) is defined in Equation 4, where ξ_{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 9 (b)). Variations in properties of the isolation system, for normal-class buildings (Class I-II, IF $\leq$ 1.0), provided that extreme (maximum or minimum) values do not differ by more than 15% from the mean values, are neglected.

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

Structural Performance Requirements

The design of both structure and 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 q (equivalent to the R in the US) 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 or detailing requirements for ductility. Design performance criteria when using ELFM 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, especially for higher importance Classes of buildings.

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 it to an external board.

Prototype and Quality Control Testing

The design of seismic isolated structures in Europe requires quality control tests to be performed on at least 20% (and a minimum of four units) of the total number of devices installed. The latter requirement is based on quasi-static procedures and they might be performed internally by the manufacturer itself.

Prototype tests, including dynamic procedures, are required to be performed on at least two units and they should be certified by an independent party and achieve CE mark approval. Whilst quality control tests are required for every project, prototype tests may not be required if the variations in properties of the devices with respect to those of previously tested units are within the ranges defined in EN 15129:2009.

Italian Code (NTC08) [5]

The Eurocode sets general criteria, while leaving a certain degree of freedom to individual countries for specifying their own requirements. In Italy the NTC08 (2008) [5] incorporates major improvements with respect to the basic EC8 requirements. The most important are:

- The adoption of a hazard map over the whole country. The most recent design code [5], building on the lessons learnt from recent major earthquakes in 1997 (Umbria-Marche) and 2002 (Molise), features a more detailed 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 depends 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 (e.g. 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;
- The introduction of a collapse limit state event (practically equivalent to a MCE) to design the isolators. The Italian code (NTC08), more recent than the Eurocode 8, sets more detailed and stringent criteria for designing the isolators which should withstand a collapse limit state event (or MCE level, rather than the DE level);
- Capacity design and minimum detailing requirements for ductility in the superstructure, imposing a minimum design acceleration for the superstructure (0.07g) and detailing of the structural members to guarantee minimum ductility capacity.

For any location around the country, the code provides specific values for defining the design spectrum at various return periods, ranging from 30 to 2475 years. The return period T_R is directly calculated (Equation 4) based on the design life (V_N), the importance level (importance level factor, C_u) of the structure and the probability of exceedance (P_{VR}) at the design level under consideration (as defined in Table 15), the designer can adopt more realistic seismic loadings, hence avoiding very large inputs in regions with low-hazard, whilst requiring larger demands in highly seismic regions and/or for important structures.

Note that for a design life of 50 years the return periods for the SLV (Life-Safety) and SLC (Collapse Prevention) are equal to 475 and 975 years, respectively, with these values increasing proportionally to the importance factor applied to the structure. However, the intensity of the design seismic event may not change linearly with the return period, but rather it will follow the local hazard at the site. This represents a significant shift with respect to other codes and arguably a significant improvement when compared to previous versions towards a more realistic representation of the design earthquake actions. As shown in Figure 8, with reference to the PGA, the seismic intensity does not vary linearly with the return period, in particular for regions with low level of seismic hazard (such as Milan in the example of Figure 8) so that, in general, the assumption that the MCE level is 1.5 times the DE level might be incorrect.

$$T_R = -V_R / \ln\left(1 - P_{VR}\right) = -V_N \times C_U / \ln\left(1 - P_{VR}\right) \tag{5}$$

$$V_R = V_N \times C_U \tag{6}$$

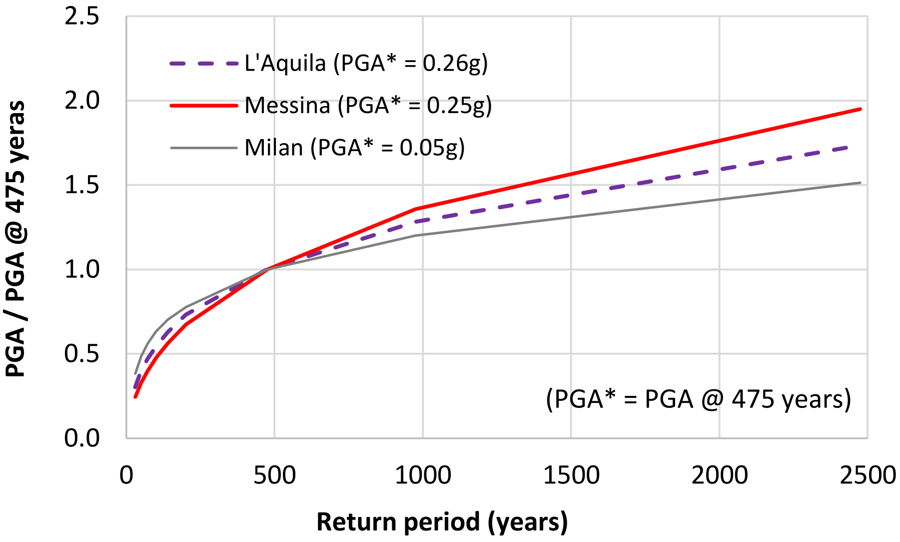


Figure 8: Seismic intensity level (PGA) vs return period (NTC 2008 [5]).

Table 15: Seismic intensity levels and probability of exceedance – NTC 2008 [5] (IT).

Limit state		Probability of exceedance (P_{VR}) within a 50 years' design life
Serviceability	SLO (Operational)	81%
	SLD (Damage control)	63%
Ultimate limit	SLV (Life safety)	10%
	SLC (Collapse prevention)	5%

In terms of isolator design, the Italian code NTC08 requires that these devices/units can sustain the displacement demand estimated for the collapse (prevention) limit state event, referred to as SLC in the code (5% probability of exceedance over the reference design life period V_R, Equation 5), which is in principle equivalent to a MCE (in the US code), although based, as discussed, on a more direct correlation with the hazard level at the specific site. For non-linear isolator devices/units, the design displacement obtained from the analysis should be amplified by adding the larger of the residual displacement at the serviceability level or damage control (SLD) and 50% of the displacement obtained from unloading the units from the SLD peak displacement conditions.

While the isolators are designed to the SLC event, the superstructure is to be designed for the Life-Safety Limit State, SLV, earthquake (10% probability of being exceeded over the reference period V_R). This approach is fully compatible with a design level earthquake in all the other codes covered in this paper. For the superstructure, the design SLV forces can be reduced by a factor of 1.5 (i.e. force reduction factor) to account for limited ductile behaviour, as in EC8, however requiring (in addition to EC8) capacity design and minimum detailing for ductility to be provided.

The modifications included in the Italian code represent a significant improvement with respect to EC8. However, as an English version of NTC08 is not available, it is likely that these refinements and improvement of design approach do not receive proper dissemination to the engineering and research community outside of Italy and references to best practice and codifications of base isolation in Europe will continue to be made, albeit, inappropriately, to the existing EC8 version only.

Qualification tests on the devices may not be required if a technical approval from the government has been already obtained.

Summary and Synoptic Comparison of International Code Provisions

From the review 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 pre-defined by the government. In Japan a simpler design process is, in theory, available for small construction, although it is understood that it is very rarely if ever used in practice. From a design perspective, the Japanese code applies amplification factors on demands and safety factors on capacities in order to reduce the computational effort (for example when considering torsional effects). Factors are adopted to introduce a safety margin on the isolators 'design towards a larger-than-design event (i.e. collapse limit or MCE-type earthquake), and a capacity protection factor (sort of overstrength factor) to 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. The damping reduction factor (Figure 9(a)) are generally lower (less conservative) than the values adopted in the US or EC8, however, as it can be observed from the case study presented in the following section, that the overall design may not necessarily end up being more conservative when compared with the other international codes.

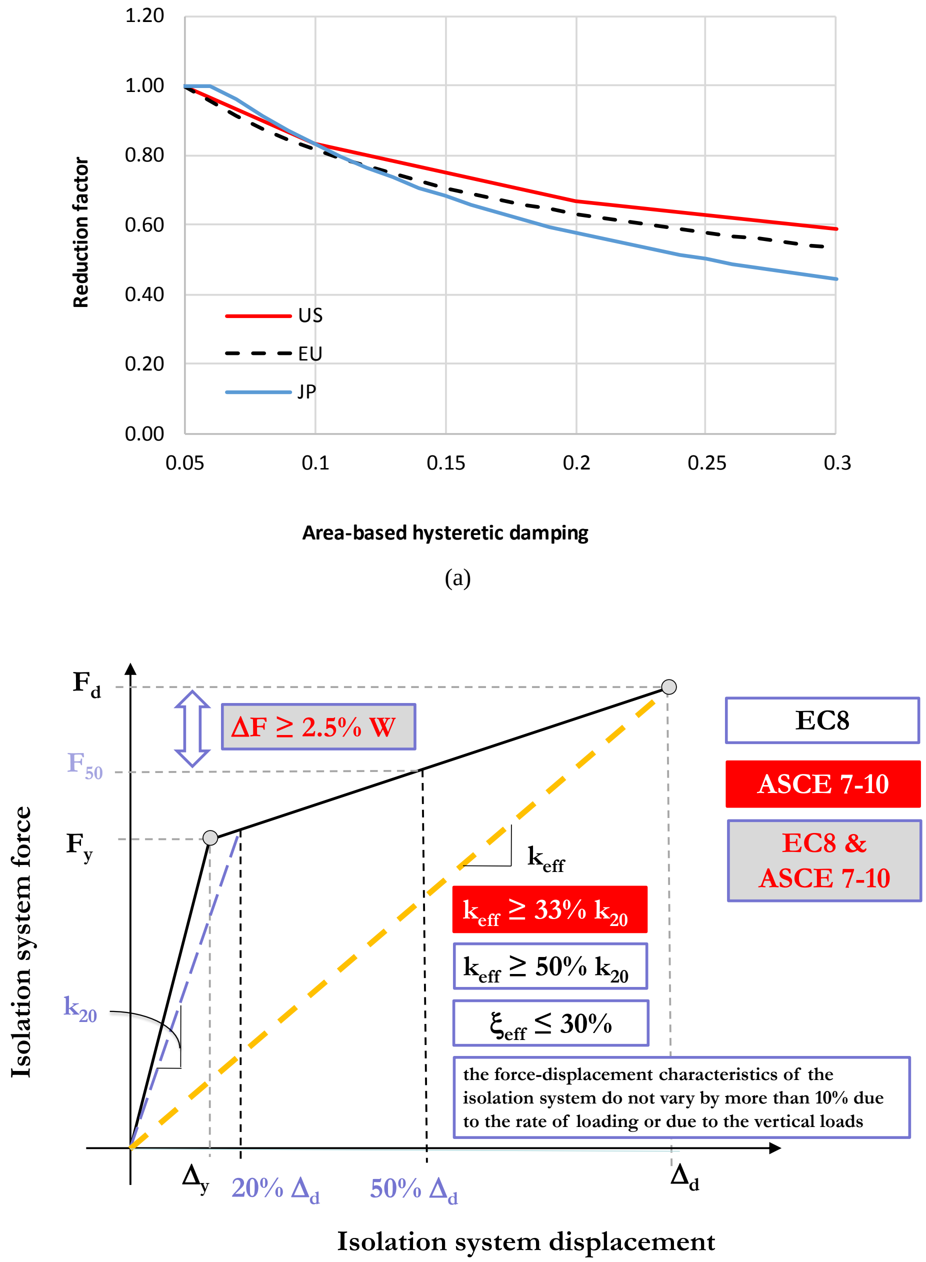
The US code, by virtue of the maximum direction MCE_R level adopted for designing the isolators, results in higher displacement demands on the devices/units, while the allowance for a certain level of ductile behaviour of the superstructure leads to lower design forces, however requiring adequate structural detailing for seismic loading. This distinguishes the US code approach from the Japanese and the EU ones, 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 to the isolator displacement demand obtained from a 500 years' design event, thereby providing 20% additional displacement capacity for a higher than DE level earthquake. Moreover, the Japanese code, introduces an amplification factor of 1.3 to the base shear, on the basis of capacity design considerations, when forces are obtained from nominal isolator properties.

It is worth reminding that the EC8 (2004) does not represent the most advanced European regulation in terms of seismic design for base isolated structures. 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 for the ASCE 7-10 approach, the Italian Code includes stringent requirements for the adoption of an equivalent linear response for the isolators, hence limiting the selection of the characteristics of the devices (i.e. force-displacement response curve). 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 in EC8 and NTC08 – proportional to the storey mass and floor displacement in ASCE 7-10), and the adoption of a structural importance level coefficient $IF \geq 1.0$ in EC8 and NTC08.

In general, it is apparent that, for ordinary buildings (Importance Factor, IF = 1), ASCE 7-10 seems to provide the most conservative design of the isolators (in considerations of the MCE displacement demand) when compared with the other codes. The design forces compared in the following case study are quite similar for the Euro and US codes. The US code requires the same ductile structural detailing for the superstructures as per conventional (fixed-base) structure.

The above 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, ASCE 7-10 -based design does not use an importance factor. Thus the displacement demands would be the same for all building types with California's hospitals limited to an R-Factor of 1.5. The design criteria for the isolators is also very different between the EU and US codes on one side and the Japanese code on the other side, with the latter allowing for more flexible systems, by virtue of setting a minimum level for the tangent (effective/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 beyond which a constant displacement assumption is made (very large corner period, in the order of 8-12 s in California - according to the US hazard map). Other codes, such as EC8 or NZS 1170.5 [14], implement a corner period (2 s in EC8, 3 s in NZS 1170.5), and hence a constant displacement demand region. In general the definition of a constant displacement plateau appears 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.



(a)

(b)

Figure 9: (a) Comparison between spectral damping reduction factors in US, European and Japanese codes, and (b) requirements for the adoption of a linear model for the isolators in ASCE 7-10 and EC8.

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Table 16: Comparison between Japanese, US and European code requirements.

	JP (MRIT)	US (ASCE 7-10)	EU (EC8)
Design methods	No Calc. / ELFM / NLTHA	ELFM / RSA / NLTHA	ELFM / RSA / NLTHA
Design Spectra (Isolators / Building)	DE/DE	MCE/DE	DE/DE
Spectrum	Dominant component	Maximum rotated component	Dominant component
Importance factor (IF)	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 ±15%
Safety factor on isolation displacement capacity	elastic ID = 0.8 – sliding/friction = 0.9	Implicit in the MCE design level	1.2
Torsion in ELFM	1.1	Calculated	Calculated
	Max ecc. 3%L	No limit	Max ecc. 2.5%L
	(+ 0% accidental ecc.)	(+ 5% accidental ecc.)	(+ 5% accidental ecc.)
Clearance	> > Max Design disp.	Max Design disp.	Max Design disp.
Building requirements	Elastic	low ductile (max R = 2)	low ductile (max R = 1.5)
	$V_{base} = 1.3\ V_{ELFM}$	$V_{base} = 1.0\ V_{ELFM,DE}$	$V_{base} = 1.0\ V_{ELFM}$
	CD and ductility req.: NO	CD and ductility req.: YES	CD and ductility req.: NO
Modelling	Simple 2D, even for NLTHA	2D for ELFM, otherwise 3D	2D for ELFM, otherwise 3D
ELFM basic conditions	T2 > 2.5 s 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
Quality Control Testing	On all devices installed	On all devices installed	On 20% of devices installed (min 4)

CASE STUDY

In order to provide a clearer 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,soil class D, hazard factor Z= 0.3 (or PGA= 0.3g) and two different Importance Levels: IL 2 (corresponding to an ordinary structure or Office building (O)), and a IL3 (Hospital facility (H)). A summary of the design parameters for the different scenarios is presented in Table 17. For the purpose of the comparison presented in this paper, the vertical acceleration component, as well local site effects and soil specific modifications are neglected.

The assumed building geometry, and isolators' design parameters (according to the ASCE and Eurocode designs), satisfy the requirements for the ELFM to be adopted. The ELFM design equations and the corresponding data for all codes are summarised in Table 17. The design is performed assuming two alternative isolator device solutions, namely Lead Rubber Bearings (LRB) alone or combined with low-friction flat sliders, or Friction Pendulum Sliders (FS).

To facilitate the comparison of the outcomes of the design approaches from different codes against a fixed set of parameters, the design is performed with reference to an isolated period T_2 =2.7 s and a yield level of the isolation system F_y = 5% and 10% of the total supported seismic weight, W, for the LRB-design and the FPS-design, respectively,. Consequently, the resulting designs might not represent the optimal solution for each particular set of devices, nor reflect the construction practice of each country.

Preliminary design values for the isolators are provided in Table 19, 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 ELFM (Table 17 and Table 18). It is worth recalling that, for the LRB, the diameter of the lead plug dictates the yield threshold, while the height of the rubber governs the post-elastic stiffness and the diameter governs the displacement capacity. For the FS, instead, 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 adopted as design solution for the layout with rubber bearings. 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. Note that variations in the stiffness of the FPS after the activation of the device are not considered, being the latter entirely governed by the radius of curvature of the concave surface.

Sample building:
Moment resisting frame building: 3 storeys
Interstorey height = 3.5 m
4 x 2 bays: 8.5 m span
Seismic weight: 13,718 kN
(Basement: 35%, 1st: 28%, 2nd: 28%, Roof: 9%)

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

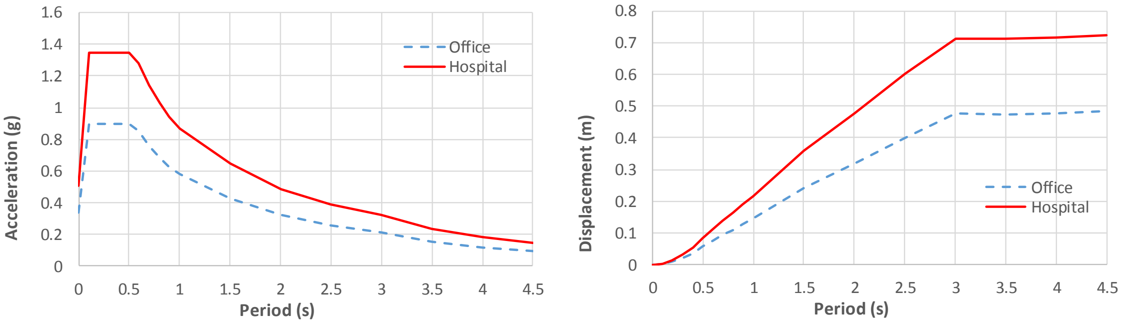

Figure 11: Design (5%-damped) acceleration response spectrum (left), and pseudo-displacement spectrum (right) – (according to NZS 1170.5, Z= 0.3, Soil type D, no Near Field Factor, IL2 or IL4).

Table 17: Case study: ELFM procedure: LRB design.

Parameter	**JP (MRIT)**	**US (ASCE 7-10)**	**EU (EC8)**
	Case Study **Office / Hospital**	**Case Study** **Office / Hospital**	**Case Study** **Office / Hospital**
Tangent period, T_2 (s)	2.7 / 2.7	2.7 / 2.7	2.7 / 2.7
Isolator Yield force ratio (Fy/W)	0.05 / 0.05	0.05 / 0.05	0.05 / 0.05
Centre of mass displacement, Δ_{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, β	0.8 / 0.8	1.0 / 1.0	1.2 / 1.2
Max design displacement, Δ_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

Table 18: Case study: ELFM procedure: FS design.

Parameter	JP (MRIT)	US (ASCE 7-10)	EU (EC8)
	Case Study **Office / Hospital**	**Case Study** **Office / Hospital**	**Case Study** **Office / Hospital**
Tangent period, T_2 (s)	2.7 / 2.7	2.7 / 2.7	2.7 / 2.7
Isolator Yield force ratio (Fy/W)	0.10 / 0.10	0.10 / 0.10	0.10 / 0.10
Centre of mass displacement, Δ_{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, β	0.9 / 0.9	1.0 / 1.0	1.2 / 1.2
Max design displacement, Δ_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 19: Case study: reference design parameters for the seismic isolation units.

Displacement Demand	LRB's - Sliders	LRB Diameter (mm)	Rubber Height (mm)	Total Height (mm)
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).*

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 full case study design exercise, initially carried out assuming an Office building (O), is then repeated assuming a higher importance class as typical of a Hospital (H). A reference IF of 1.4 is adopted in both the EC8 and Japanese Code designs, while no variations (if minimum by codes are adopted) according to the US Code, which does not explicitly incorporate any Importance Level or Factor.. The design spectra for the "Office-type" building scenario and the "Hospital-type" scenario are represented in Figure 11.

A summary of the major properties of the designed isolators is provided in Table 19. It is worth noting that:

1. The ASCE 7-10 design accounts for an MCE type event (here assumed as 1.5 the reference IL2 design spectrum) and does not change in the case of Hospital building (H) when compared to the Office building (O);
2. The Japanese design allows for the adoption of more "flexible" systems as there are no o limitations on the linear modelling of the devices.

From the analysis of the results obtained for this simple case study (Table 17, Table 18 and Table 19), it is observed that the EU and US code designs result into similar levels of design forces for the superstructure, the only major difference being lower displacement demand required for an office-type building (O) according to the EC8-design. Consideration of a MCE level, linear modelling requirements for the isolators and the introduction of an importance factor for the building are the primary sources of difference between the EU and US approach. Also, the shape and the definition of the design spectra also play an important role if these codes were directly implemented in other countries.

For the above considerations, the application of overseas codes for the design of base isolated buildings in New Zealand, in the absence of a dedicated standard or design guidelines, would suggest to inevitably lead to different designs and hence building actual performances, with consequent lack of homogeneity between structures of the same class. Furthermore, full consistency ad compatibility with the other New Zealand standards might not be easily guaranteed not reliably checked, 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 structural performance criteria and requirements.

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

1. Design spectral shape: NZS 1170.5(2004) [14] implements a constant displacement region for periods longer than 3 s, which is potentially un-conservative for the design of base isolated systems (typically dealing with longer periods in the 4 to 6 second range) unless 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) [15], produced new hazard maps for the country in terms of horizontal displacement response spectral ordinates for a wide range of vibration periods, from 0.05 s up to 20 s, 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 a S_p factor (as defined in NZS1170.5) when dealing with base isolated structures: careful considerations should be given in particular when considering the evaluation of the design displacement (demand) on the isolators. While all the international codes reviewed in this study 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 NZS1170.5 [14] standard, to the design displacement (demand). In fact some codes apply magnification factors on the design displacement of the isolators.
3. Criteria requirements for a local seismic hazard study to be performed, i.e. under which circumstances and depending on the design approach.
4. Design levels (DE and/or MCE?), importance factors and additional safety/magnification factors that should be used; these may differ for IL2 (ordinary) and IL4 (critical facilities) buildings.
5. Design boundaries on the adoption of simplified Equivalent Lateral Force Methods (ELFM).
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 isolator properties.
9. Ductility and force reduction factors for the superstructure base shear and drift/ductility limits for the superstructure response.
10. Structural detailing requirements, i.e. 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.

CONCLUSIONS

The overview of three major international codes for the design of base isolated buildings has outlined the current differences 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 New Zealand 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 the authors' opinion that the engineering community in New Zealand should collaborate at national and international level towards the development of a New Zealand-specific design "code", or endorsed guidelines, for base-isolated buildings. Such document could bring together the current best practice in New Zealand and overseas, whilst assuring consistency with the current standards as well as homogeneity in the design of base-isolated structures throughout the country.

Note: at the time of publishing this paper a NZSEE Working Group has been recently established and work is in progress to develop design guidelines for base-isolated buildings in New Zealand, with the support and endorsement of MBIE (Ministry of Business, Innovation and Employment) and EQC (Earthquake Commission)

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