# Performance Criteria and Design Parameters

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**In the proposed seismic action model to be incorporated into the National Annex (NA) to Eurocode 8 (EC8) for Malaysia, the design seismic actions for important built facilities are benchmarked on a 2,475 years return period (RP) earthquake action whereas the reference seismic action (notional 475 year RP) to be considered for ordinary buildings is the design action scaled by a factor of 2/3. Decisions leading to the proposal are explained and terminologies clarified in the paper which draws frequent references to the literature citing major codes of practices.**

## KEYWORDS
National Annex to Eurocode 8, Seismic Design Actions.

## INTRODUCTION

The decision on the return period of the design seismic actions and the resulting design peak ground acceleration value for buildings of different importance classes in different parts of Malaysia, is a major item of consideration to be discussed in this paper along with performance criteria for buildings of different classifications.

## STRUCTURAL PERFORMANCE CRITERIA AND PARAMETERS FOR DESIGN SEISMIC ACTIONS

### *Performance Criteria*

According to EC8 – Part 1 (CEN, 2004), building structures shall be designed and constructed in such a way that the requirements of (i) No Collapse (NC) and (ii) Damage Limitations (DL) are met. The state of No Collapse is essentially in alignment with designing to the ultimate limit state which entails the protection of life in a rare earthquake event, by ensuring that no part of the structure collapses and that adequate residual lateral resistant capacity of the structure remains after the event to withstand strong aftershocks should these occur. The safety of the occupants can be assured, but the built facility can be inhabitable or the damage can be too costly to repair.

The "no collapse", or "no local collapse", design criterion as described, is comparable to the "life safety" performance criterion as defined in *SEAOC Vision 2000* document (SEAOC, 1995) in the United States and the "significant damage" (SD) performance criterion stipulated in EC8 – Part 3, which contains provisions for the seismic assessment and retrofitting of existing buildings. The No Collapse performance criterion is not to be confused with the "near collapse", or "collapse prevention", performance criterion of *SEAOC Vision 2000* which is about ensuring that the building is able to sustain sufficient vertical load carrying capacity in a very rare earthquake event when the structure is on the verge of wholesale collapse with little or no residual lateral resistance, and some falling hazards may be present (Booth, 2014; Fardis, 2009).

The Damage Limitations (DL) performance criterion, which corresponds to the service ability limit state criterion (in the conventional limit state design approach), has also been written into both Part 1 and Part 3 of EC8 and is intended to address the damaging potentials of frequent or occasional earthquake events in the design of ordinary buildings. The DL performance criterion is comparable to the "immediate occupancy", or "operational", performance criterion of *SEAOC Vision 2000* which is to ensure no permanent drift and no loss of lateral strength and stiffness of the building structure. The built facility is then fit for continuous occupation during the recovery period and the functionality of the building will not be interrupted significantly by repair activities. In regions of low or moderate seismicity that are remote from tectonic

*Table 1: Performance Criteria of Building Structures*

| No. | Eurocode 8 part 1 | Eurocode 8 part 3 | SEAOC Vision 2000 | Descriptions |
|---|---|---|---|---|
| 1. | - | - | Fully Operational | Components that are sensitive to drift and/or acceleration remains fully functional in a frequent event. |
| 2. | Damage Limitation | Damage Limitation | Operational or Immediate Occupation | No permanent drift and no loss of lateral strength or stiffness of the building. The built facility remains to be fit for continuous occupation in an occasional event. |
| 3. | (DL) | Significant Damage | Life Safe | No part of the structure collapses and adequate residual lateral resistant capacity remains in the structure after a rare event to withstand strong aftershocks in order that safety of the occupants can be secured but building may be inhabitable and repair too costly. |
| 4. | No Collapse | Near Collapse | Collapse Prevention or Near Collapse | Structure is able to sustain sufficient vertical load carrying capacity in a very rare earthquake event when the structure is at the edge of wholesale collapse. Residual lateral resistant capacity of the building might have been lost. |

plate boundaries, only rare or very rare earthquake events are of concern. So, the DL performance criterion need not be checked in such an environment except for built facilities forming part of lifeline facilities in the aftermath of an earthquake disaster or buildings containing hazardous materials.

Refer to Table 1 for a summary of the performance criteria of building structures as defined by the two parts of EC8 and the *SEAOC Vision 2000* document.

**Parameters for design seismic actions**

In this section, recommendations for the value of the return period of seismic actions and PGA values for buildings of different importance classes and the behaviour factor are discussed. The return period of the considered seismic actions that are aligned with the No Collapse (NC) performance criterion, is to be decided on a country-by-country basis, given that factors governing such a decision would involve social, economic and political considerations. Thus, the return period for the NC performance criterion is to be specified in the respective NA of the country.

It is stated in the footnote attached to Clause 2.1 in EC8 – Part 1, that ground motion intensity in a rare earthquake event consistent with a 10% chance of exceedance for a design life of 50 years (i.e. return period of 475 years) is recommended as the design seismic action. It was noted that this recommendation was drafted in the late 1990s, at a time when it was still the norm to not consider return periods exceeding 475 years in the design of structures supporting ordinary buildings (Booth, 2014). Implicit in the NC performance criterion is that the building is expected to have sufficient additional reserve capacity to sustain a very rare, and extreme, earthquake event without experiencing wholesale collapse (Fardis, 2009).

Seismic design provisions around the world have evolved over the decades, during which time experience gained through field observations from places like California, have been taken into account in numerous code revisions. In such an environment dominated by active faults, the intensity of ground shaking is increased by a factor which is slightly greater than 1.5 as the return period is increased from 475 years to 2,475 years (Tsang, 2014). Code compliant constructions that have been designed to fulfil NC performance criterion are expected to have sufficient additional reserve capacity to also fulfil collapse prevention criterion when subject to seismic actions that are 1.5 times the design level. Despite this margin of safety from collapse that is implicit in contemporary practices, major earthquake disasters in recent years, including the 1995 Kobe earthquake in Japan and the 2008 Sichuan earthquake in China, prompted a critical review of the adequacy of this long established convention of designing to a return period of 475 years (Tsang, 2011).

In regions of low or moderate seismicity (where earthquakes occur infrequently and active faults are difficult to identify), ground shaking intensity ratio that is associated with an increase in return period from 475 years to 2,475 years, can be escalated to a value much greater than 1.5. A factor varying between 2.4 and 5 is predicted for earthquakes in an intraplate environment (Tsang, 2014, Geoscience Australia, 2012). Given these predictions, building structures designed on a return period of 475 years to fulfil NC performance criterion in an intraplate environment, would not automatically possess adequate additional reserve capacity to prevent collapse in a very rare event.

The trend of moving away from the conventional practice of designing to a return period of 475 years was initiated by the influential FEMA450 document (BCCS, 2003) which was to guide the design of new buildings in the United States. The design seismic action was recommended to be based on a maximum considered earthquake (MCE) of 2,475 years, scaled down by a factor of 2/3 (reciprocal of 1.5). This scaling factor can be interpreted as the margin between the state of NC and collapse prevention of the structure in order that code compliant buildings can always be assured of the capacity to prevent collapse in a very rare earthquake event.

The 2005 edition of the National Building Code of Canada (NRCC, 2005) increased the return period from 475 years to 2,475 years without applying a scaled down factor of 2/3 (Mitchell *et al.*, 2010) but a generous 2.5% drift limit, which was consistent with the Collapse Prevention performance criterion, was specified. The

NA to EC8 for the United Kingdom (BSI, 2008) also specified a return period of 2,475 years to override the recommendation of 475 years in EC8 – Part 1 (CEN, 2004) for designing to No Collapse (Life Safe) performance criterion, which was more stringent than requirements in Canada.

In perspective, a design return period of 2,475 years is actually not overly conservative, given that the annual fatality risk of an occupant in a building which has been designed to a return period of 2,475 years is of the order of 10-6, which is consistent with involuntary fatality risk affecting building occupants in other types of natural disasters (Tsang, 2014).

In view of the facts presented in the above design, seismic actions presented in terms of PGA values on rock sites are recommended herein for various importance classes of buildings as summarised in Table 2 for Peninsular Malaysia, Sarawak and Sabah. It is shown that all built facilities of importance class IV, including hospitals, emergency services and other lifeline facilities, are to be designed to a return period of 2,475 years to fulfil NC performance criterion in order that these facilities are safe to occupy in the aftermath of a very rare event as well as fit to continue to operate in more frequent events. Reference seismic actions to be considered in the design of ordinary buildings of importance class II in Peninsular Malaysia and Sarawak, are accordingly based on a reference PGA value of 0.07g (being 0.1g/1.5) which provides adequate protection of ordinary buildings from collapse in a very rare earthquake event. By interpolation a design PGA of 0.08g is stipulated for buildings of intermediate class III such as condominium, schools and public buildings which can house a large number of occupants at times.

*Table 2: Design PGA on rock sites for Peninsular Malaysia, Sarawak and Sabah.*

| Importance Class | Importance Factor, $\gamma_I$ | Recommended Building Categories | Notional design PGA, $a_g$ (g's) | |
|---|---|---|---|---|
| | | | Peninsular Malaysia and Sarawak | Sabah |
| I | 0.8 | Minor constructions | 0.06 (0.8 x 0.07) | 0.10 (0.8 x 0.12) |
| II | 1.0 | Ordinary buildings (individual dwellings or shops in low rise buildings) | 0.07 Reference PGA (notional 475 years RP) | 0.12 Reference PGA (notional 475 years RP) |
| III | 1.2 | Buildings of large occupancies (condominiums, shopping centres, schools and public buildings) | 0.08 (1.2 x 0.07) | 0.14 (1.2 x 0.12) |
| IV | 1.5 | Lifeline built facilities (hospitals, emergency services, power plants and communication facilities) | 0.10 (2,475 years RP) | 0.18 (2,475 years RP) |

Seismic actions to be considered for design purposes for any building class at any location in Malaysia are to be derived from the benchmark model based on a return period of 2,475 years and then scaled down in accordance to the respective design PGA value as listed in one of the tables. The allowed inter-storey drift limit is 1.5% to fulfil NC, or life safe, performance criterion.

The proposed seismic actions to be considered for the design of built facilities for Peninsular Malaysia and Sarawak are less stringent in many ways than those adopted in Canada and in the United Kingdom where ordinary building structures are to be designed to a return period of 2,475 years, and can be described as comparable to the planned revision to the Australian Standard which stipulates a minimum design PGA value of 0.08g for ordinary buildings irrespective of results from updated probabilistic seismic hazard analyses.

Finally, a behaviour factor ($q$) is to be stipulated to take into account the capacity of the structure at the member level to withstand seismic actions beyond its notional capacity limits. The elastic spectrum is to be scaled down by the factor of $1/q$ into the design spectrum (refer Clause 3.2.2.5) for linear analysis, from which the displacements shall be multiplied by the displacement behaviour factor $q_d$ (assumed equal to $q$ unless otherwise specified) (refer Clause 4.3.4).

For damage limitation requirement (i.e., level 2 in Table 1), while it is deemed to satisfy for Class I to III buildings, only Class IV buildings need to be checked in the calculation of interstorey drifts $d_r$ (or deformation). With lifeline facilities such as a hospital (a class IV building), non-structural installations must also be designed to a RP of 475 years (i.e., 10% probability of exceedance in a life span of 50 years) to ensure that the functionality of the facility is not significantly compromised by earthquakes. This level of ground shaking is not to be confused with that used for checking NC compliance of Class II structures, which is based on a notional RP of 475 years (being 2/3 of the intensity associated with a RP of 2,475 years by definition). The reduction factor for displacement of $\nu = 0.5$ is to take into account the difference between the two levels of intensities (refer Clause 4.4.3.2).

In the Australian Standard (AS1170.4, 2007), the additional capacity to withstand seismic actions is resolved into the performance factor ($S_p$) which takes into account contributions from the over-strength of materials and the structural system as a whole in sustaining earthquake generated lateral forces whereas the ductility ratio ($\mu$) takes into account contributions from the ability of the structure to deform in a ductile manner (AEES, 2009). The value of $S_p$ is taken by default as 0.77 and the value of $\mu$ is taken as 2.0 by default for limited ductile reinforced concrete, structural steel or composite structures which employ concrete and steel as construction materials. The composite factor of 2.6 (being $\mu/S_p$ or 2/0.77) that is used as default design value in Australia can be compared to a slightly lower, more conservative, $q$ value of 2.0 recommended in the *National Building Code of Canada (NBCC)* since its 2005 edition. Given that the default $q$ value stipulated in the NA for Singapore is 1.5 which is consistent with recommendations by EC8, members of the study group have agreed to this figure for use in Malaysia, pending further studies in the future to justify a higher value. A local study (Chiang *et al.*, 2012) revealed that the mean strength to characteristic strength ratio of thousands of concrete cube tests up to grade C40 in Malaysia was 1.2, which justified the recommendation for over-strength factor. The inherent ductility of concrete structures is conservatively assumed as 1.25 to arrive at a $q$ value of 1.5 in totality. The recommended and default values of $q$ that is stipulated in regulatory documents in countries of low to moderate seismicity for limited ductile structures are listed in Table 3.

*Table 3: Recommended and default values of behaviour factor q for limited ductile structures.*

| Region/ Country | Standards/ Codes | Over-strength factor | Ductility | Behaviour factor |
|---|---|---|---|---|
| Malaysia | Proposed NA to *MS* | 1.2 | 1.25 | 1.5 |
| Europe Singapore | Eurocode 8 NA to *SS* | 1.5 | 1.0 | 1.5 |
| Canada | NBCC | 1.3 | 1.5 | 2.0 |
| Australia | AS1170.4 | 1.3 | 2.0 | 2.6 |

## COMMENTS ON THRESHOLD OF LOW SEISMICITY

EC8 recommends an upper threshold value of $a_g$ = 0.78 m/s² for low seismicity, which is based on a RP of 475 years. As the hazard level of Malaysia is benchmarked on a 2,475 year RP, such threshold value has been scaled up by the actual demand ratio of RP 2,475 years to 475 years which is equal to 2.4 (Lam *et al.*, 2015). Hence, a value of $a_g$ = 1.87 m/s² for a RP of 2,475 years shall be adopted as the upper threshold value for low seismicity, while the whole of Malaysia can be classified as low seismicity.

EC8 recommends an upper threshold value of $a_g$ = 0.39 m/s² for very low seismicity, which is based on a RP of 475 years. Likewise, a value of $a_g$ = 0.94 m/s² for a RP of 2,475 years can be adopted as the upper threshold value for very low seismicity. Hence, no part of Malaysia is classified as very low seismicity. In other words, no parts of Malaysia should be put into the "no requirement for seismic design" category. In an intraplate region like Malaysia, areas that have never experienced earthquake tremors should not be automatically declared free of local earthquakes in the future. That is an unsafe assumption to make.

## CONCLUSION

i. Lifeline facilities, including hospitals and infrastructure in support of emergency services, are to be designed to fulfil "no collapse" (life safe) performance criterion for a return period of 2,475 years. Lower design seismic actions are recommended for buildings of other importance classes.
ii. Response spectrum to be used for design purposes, is scaled in accordance with the considered notional design peak ground acceleration values on rock sites which vary between 0.06g and 0.10g for Peninsular Malaysia and Sarawak and between 0.10g and 0.18g for Sabah for various importance classes and return periods. Exact values as presented in the tables depend on the importance classification of the building.
iii. The allowable inter-storey drift limit to satisfy no collapse criterion is recommended to be 1.5%.
iv. Design actions at the member level, such as bending moments and shear forces, are to be scaled down by $1/q$ where $q$ is the behaviour factor. Members of the study group have agreed to the default value of 1.5 consistent with practice in Singapore but larger values could be adopted. The default values adopted in Canada and Australia are higher.


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### *Notations*

$S_p$ performance factor
$a_g$ notional design peak ground acceleration on rock
$d_r$ design interstorey drift
$q$ behaviour factor
$q_d$ displacement behaviour factor
$\gamma_I$ importance factor
$\mu$ ductility ratio
$\nu$ reduction factor for interstorey drift limit associated with the damage limitation requirement. ■

## REFERENCES


[1] AS 1170.4 (2007) Structural Design Actions – Part 4 Earthquake Actions. Standards Australia.

[2] AEES (2009) AS 1170.4 Commentary: Structural Design Actions – Part 4 Earthquake Actions. Victoria: Australian Earthquake Engineering Society.

[3] BC3 (2013) Guidebook for Design of Buildings in Singapore to Design Requirements in SSEN-1998-1. Singapore: Building and Construction Authority.

[4] Booth, E. (2014) Personal communications in December 2014.

[5] BSI (2008) NA to BS EN1998-1: 2004 UK National Annex to Eurocode 8: Design of Structures for Earthquake Resistance. Part 1: General Rules, Seismic Actions and Rules for Buildings, British Standards Institution (BSI), London, U.K.

[6] CEN (2004) EN 1998 1. 2004. Eurocode 8: Design of Structures for Earthquake Resistance – Part 1: General Rules, Seismic Actions and Rules for Buildings. European Committee for Standardisation, Brussells.

[7] Chiang, J.C.L., Tu, Y.E., Tan, C.S. (2012), Gauging the reliability of structural design for buildings and infrastructures from Malaysian Engineers' viewpoint, The Twelfth East Asia-Pacific Conference on Structural Engineering and Construction (EASEC-12), Hong Kong Special Administrative Region, China, 24-26 January 2011.

[8] Fardis, M.N. (2009) Seismic Design Assessment and Retrofitting of Concrete Buildings based on EN – Eurocode 8, Springer.

[9] Lam, N.T.K., Lumantarna, E., Tsang, H.H., Wilson, J.L. (2015). Results of probabilistic seismic hazard analysis assuming uniform distribution of seismicity. Proceedings of the 10th Pacific Conference on Earthquake Engineering, 6 - 8 November 2015, Sydney, Australia.

[10] Mitchell, D., Paultre, P., Tinawi, R., Saatcioglu, M., Tremblay, R., Elwood, K., Adams, J., and DeVall, R. (2010) "Evolution of seismic design provisions in the National building code of Canada" Canadian Journal of Civil Engineering. 37: 1157-1170.

[11] NA to SS EN 1998 1. 2013. Singapore National Annex to Eurocode 8: Design of Structures for Earthquake Resistance – Part 1: General Rules, Seismic Actions and Rules for Buildings. Singapore: SPRING Singapore.

[12] NRCC (2005) National Building Code of Canada, Associate Committee on the National Building Code, National Research Council of Canada, Ottawa, ON.

[13] SEAOC (1995) Vision 2000: Performance-Based Seismic Engineering of Buildings, Structural Engineers Association of California Sacramento, California, U.S.

[14] Tsang, H.H. (2011) "Should we design buildings for lower-probability earthquake motion?" Natural Hazards 58: 853-857.

[15] Tsang, H.H. (2014) "Seismic Performance Requirements and Collapse Risk of Structures" Proceedings of the Annual Seminar entitled "Advances in Seismic Engineering" HKIE/IStructE Joint Structural Division. 58-75.