Title: **CTBUH Recommendations for the Seismic Design of High-Rise Buildings**

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Introduction and Background

Overview and need for this document

There is a resurgence in the construction of high rise buildings around the world, including in many seismically active regions. There are major differences in the approach taken to seismic design in different national jurisdictions. Whilst rigorous performance based assessments are required in Japan and China, many other nations do not require anything other than a 'UBC-style' design based on code spectra and conventional force reduction factors. Some of these simple approaches lead to unsafe and/or uneconomic designs.

The objective of this Guide is to set out best practice for the seismic design of high-rise buildings for use in any seismic region of the world.

UBC 97 and IBC 2003 are widely referenced in the design of high-rise buildings in many countries, and previous editions of the UBC still form the basis for several other national codes. These codes are ***not suitable*** for the following reasons:

1) They were developed for application to the more common building types in California 30-50 years ago, and not for the very tall modern buildings now being built around the world

2) These codes permit only a ***dual system*** for buildings above 49m in height. This is restrictive, uneconomic and unnecessary in many cases

3) A ***dual system*** is allocated a force reduction factor of 8. This is dangerous as the implied level of ductility cannot usually be achieved under the very high gravity stresses at the bottom of high-rise buildings. Rules appropriate at 49m are not necessarily valid at 200m + in height

4) Code design spectra are generally defined only up to periods of 2 - 3 seconds. For ultra-high rise structures the first mode periods can be from 5 to 10 seconds

International Context

A performance based philosophy is consistent with the best international practice, and has been explicitly required by the regulatory authorities in Japan for buildings exceeding 60m since 1981. In China, the height limits on tall buildings are set out in the Chinese code for seismic design of buildings GB50011 – 2001 and depend on the seismic zone, the structural material and the structural systems adopted. For instance, in the highest seismic zone (seismic intensity 9) that is equivalent to UBC zone 4, the tallest height limit on reinforced concrete structures is 80 m (tube-in-tube system) and that on steel structures is 180 m (tube type systems such as framed-tube, tube-in-tube and multiple-tube systems). In both countries, the performance-based approach must be adopted to demonstrate satisfactory seismic performances and the design must pass an expert panel review process for approval for buildings exceeding code limitations.

Building codes in the United States such as the Uniform Building Code (ICBO, 1997) and International Building Code (ICBO, 2003) and other national codes based on these permit performance-based approaches to design but provide little to no guidance for such design. United States building codes were developed for low- and medium-rise construction with little consideration for high-rise structures for which higher modes can dominate response and suitable framing systems differ from low-to-medium rise buildings. As a result, the direct adoption of the standard code-based design procedures can lead to relatively uneconomic structural designs and in some cases, to framing systems that will not perform well in earthquakes. Much of the framework for performance based design in the USA is now well developed in SEAOC Vision 2000, FEMA 356 and ATC 40.

The methods of Eurocode 8 are not performance based, and are not well suited to high-rise construction.

Performance based methods force the designer to assess how the building will perform under earthquakes and therefore will identify unsafe designs. At the same time they enable arbitrary restrictions to be removed and the scope for more cost-effective designs to be developed.

Effects of Seismic Zones

There are two principal qualitative differences between seismic hazards in regions of 'low seismicity' relative to regions of 'high seismicity'. These are:

1) There is a more rapid reduction in PGA with reducing return period in regions of lower seismicity. This is illustrated in figures 1.1 and 1.2.

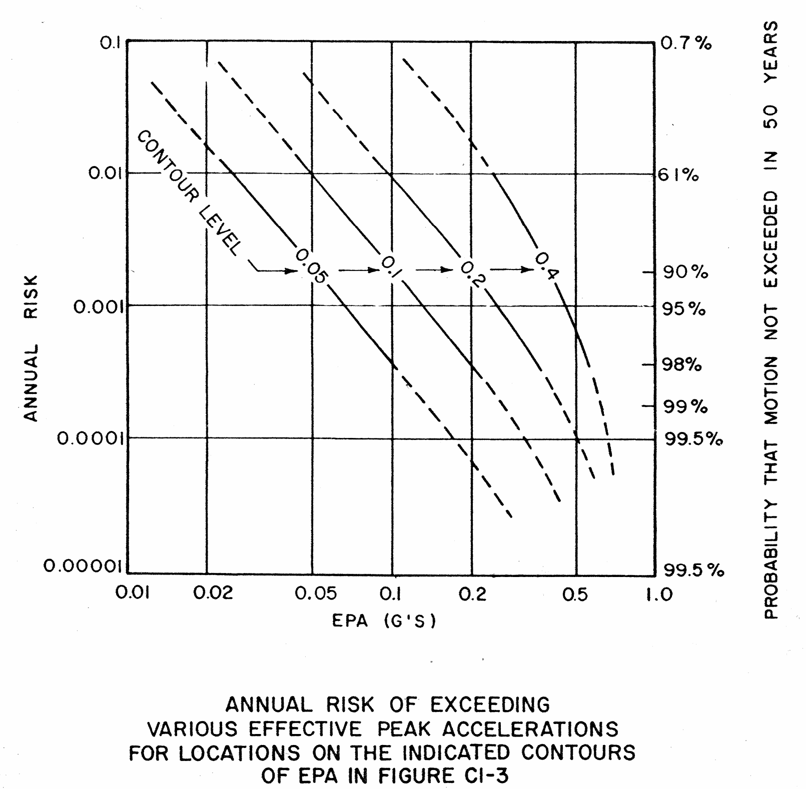

Figure 1.1 (From ATC3-06)

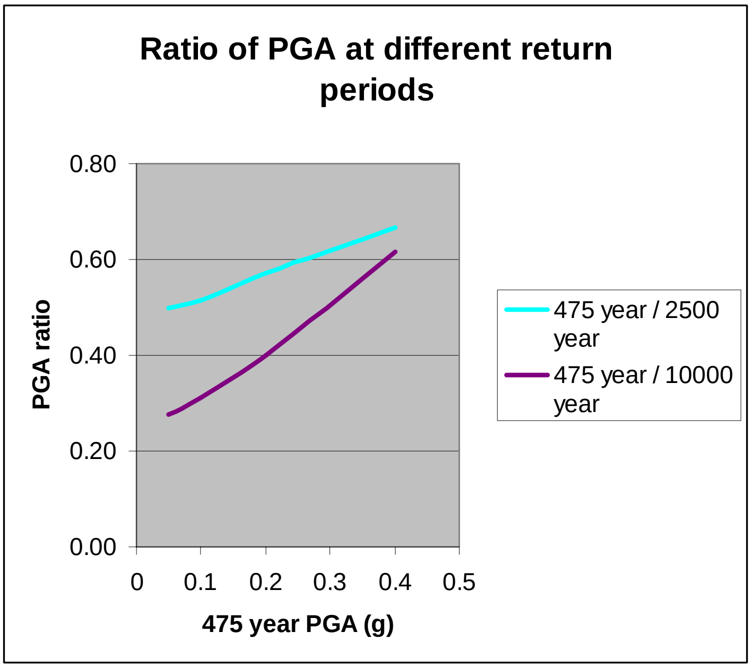

Figure 1.2 (derived from figure 1.1)

Figure 1.2 shows that whilst the PGA at the traditional reference return period of 475 years is about 60% to 67% of an MCE defined with 2500 or 10,000 year return period in a typical 'Zone 4' location, this ratio is much lower for regions of lower seismicity. This means that:

a) 475 years is too low a return period for 'life safety' assessment in regions of lower seismicity
b) Maybe 2500 years is too low a return period for the MCE in regions of lower seismicity.

1) Long period motions, and therefore spectral displacements, are relatively lower in regions of low seismicity. This is significant for high-rise structures which have longer periods. ***To be developed***

Scope – What do we mean by high-rise?

To be developed….

Design Objectives and Philosophy

Performance Objectives

Potentially critical issues for the seismic design of high-rise buildings include:

1) high overturning moment and foundation issues,
2) development of ductility in elements at the base of a structure under high compressive gravity stress,
3) controlling inter-storey drift,
4) controlling damage so as to permit repair

The normal seismic design objectives for a commercial high-rise building will correspond to the 'Basic Objective' of FEMA 356. A two level check is recommended:

1) Life safety in the design basis earthquake (DBE) having spectral ordinates 67% of those of the maximum considered earthquake.

2) Collapse prevention in the maximum considered earthquake (MCE): spectral ordinates associated with a 2475 year return period.

Japanese practice requires a three level check as follows:

1) Level 1 event (essentially elastic response under input excitation corresponding to a once-in-a-lifetime event, return period 30 – 50 years).

2) ????????

3) Level 3 event corresponding to a maximum earthquake for which no part of a building should collapse (collapse prevention objective).

The approach in China is nearly identical to that in Japan but with the Level 1 event clearly defined as that associated with a 50 year return period (63% probability of exceedance in 50 years).

Return periods and performance levels (Dual level check) *like Japanses*

Deformation based philosophy

Peer Review

Independent project-specific peer review (by an Expert Panel) is required for high-rise buildings in Japan and China if code limits on height or regularity are exceeded. FEMA 356, the 1997 UBC and the 2003 IBC also require an independent peer review by an engineer having extensive relevant experience and knowledge if nonlinear response-history analysis is adopted for performance assessment (as we recommend).

Construction Supervision

Seismic Hazard Assessment

Site specific seismic hazard assessment is recommended, particularly in locations where the extent of codified guidance is limited. Code spectra are focused at the more commonplace natural periods and site specific studies are useful in developing better data in the long period range.

If three sets of earthquake histories are used for analysis and design, the maximum of the peak responses should be used for design; if seven or more sets of earthquake histories are used, the arithmetic average of the peak responses results can be used for design.

Requirement for Hazard Assessment

-treatment of long period motions

-near fault effects

-treatment of uncertainty and variability

Probabilistic and Deterministic methods

Site response

Selection and modification of time histories

A minimum of 3 independent two-component earthquake-history sets should be developed based on real records modified by accepted methods to match more closely the desired response spectrum. Selection of recorded time histories and modification of these to match the target design spectrum should be carried out following guidelines in the widely referenced research report by the Pacific Earthquake Engineering Research Centre (PEER 2001, Ground Motion Evaluation Procedures for Performance-Based Design).

Foundation Effects

Treatment of basements and foundations
Effective level of excitation
Soil-structure interaction
Geotechnical parameters (material 'partial factors')
Rocking

Structural Analysis

Analysis requirements for dual level check

-Linear level 1 check (design properties)

Performance-based criteria are stated in terms of deformation measures (e.g. plastic hinge rotation and inter-storey drift), and for long period structures elastic seismically induced deflections are similar to those that would be obtained by non-linear analysis of the same structural model having limited element strengths. Therefore, response-spectrum analysis gives a reasonable initial estimate of the deflections expected by non-linear response-history analysis.

-Non Linear level 2 check (expected properties)

Basic Modelling Approach

Mass
Stiffness (especially RC)
Damping

Damping of high-rise buildings is lower than values (typically 5% of critical) traditionally assumed for seismic response analysis. Data from Tamura shows that whilst 5% of critical may be appropriate for concrete buildings up to about 50m, it is too high for steel buildings and taller concrete buildings. There is very little data above 200m, but it is clear that the damping of buildings of this height is nearer 1% of critical.

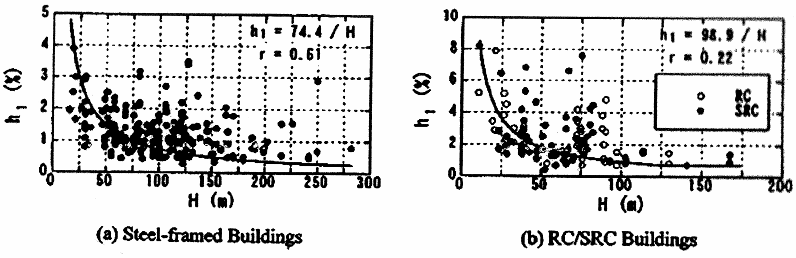

Fig. 4. Building height H versus translational first-mode damping ratio h_1

Finite joints
Coupled shear walls
Foundations

Elastic multi-mode response spectrum analysis
Non-linear dynamic procedure

Static push-over analysis, whilst having some value for low-rise regular structures, is not suitable for high rise buildings because:

1) no static lateral load distribution can represent the multi-mode dynamic load effects that arise in these structures

2) first-mode response does not dominate the total response.

The limitations are spelt out in FEMA 356 Section 2.4.2.1, based on the contribution of higher modes to the elastic response. If these checks are followed through it is invariably found that the method is inapplicable to high-rise structures because of the significance of higher mode responses.

In addition, the method is unsuitable for torsionally susceptible structures, for buildings in which bi-directional excitation places significant non-linear demand on any single element of the structure (e.g., corner columns, concrete cores) and for structures with seismic isolation or energy dissipating devices.

Therefore the non-linear dynamic procedure is required.

Non-linear modelling procedures (similar to FEMA 356)
Modelling of reinforced concrete shear walls and core walls (axial/flexure interaction)

Material Properties and modelling

Structural Steel

In general the modelling of steel elements is relatively straightforward, except for the representation of the cyclic buckling of braces. For nonlinear modelling of post buckling strength degradation as a function of axial shortening in braces, the MAT_STEEL_BRACE material model in LS-DYNA is recommended.

Reinforced Concrete
Properties of different strengths (including high strength)
Cracking
Confinement

Reinforced concrete elements are more difficult to model due to the changing stiffness of members as cracks open and close, the strength degradation as yielding proceeds, and due to the axial/biaxial bending interaction yield surface. The effective flexural stiffness is a function of the reinforcement ratio and the axial stress on a section. A useful chart to estimate the appropriate initial effective stiffness is figure RC3-4 of ATC-32.

For elements that fail in flexure and exhibit stable hysteresis the MAT_SEISMIC_BEAM material model of LS-DYNA enables the M_{yy}-M_{zz}-N yield interaction surface of reinforced concrete beam-column sections to be specified. The XTRACT software may be used to predict the interaction surface, taking account of longitudinal reinforcement and confinement to concrete due to the transverse reinforcement.

For elements that fail in shear, such as squat shear walls, piers and in some cases diaphragms and beam/column joints there is little ductility available and strength degrades as the element is subjected to repeated deformation cycles. In LS-DYNA such elements can be modelled as shells with the MAT_SEISMIC_SHEAR_WALL material definition.

High strength concrete with $f_c^{'}$ greater than 45 MPa (6.5 ksi) has traditionally not been commonly used in seismic zones because of increased brittleness (Priestly et al, 1996). The Chinese code GB50011 – 2001 specifically limits the concrete strength to 60 MPa for buildings in zone 9 (equivalent to UBC zone 4) and to 70 MPa for buildings in zone 8 (equivalent to UBC zones 2B and 3), although no specific guidance on the use of high strength grade 60 and grade 70 concrete has been provided.

SRC (Steel reinforced concrete)

Composite structural steel and reinforced concrete construction, such as reinforced-concrete-encased composite elements, concrete-filled composite elements, composite shear walls with structural steel sections, and composite shear walls with steel plates, is increasingly popular in high-rise buildings. Part II "Composite Structural Steel and Reinforced Concrete Buildings" of the 2002 AISC "Seismic Provisions for Structural Steel Buildings" SRC composite elements provides seismic design guidelines for composite elements and structures and should be consulted when these are used.

Modelling of composite elements in high-rise buildings may be carried out in a similar manner to that of reinforced concrete elements. The axial-flexural strength capacities, such as the P-M interaction yielding curves and the bi-axial moment capacity orbit curves should be calculated using a cross section analysis program, such as XTRACT or Oasys ADSEC that employs the fibre cross section model and permits the user to specify stress-strain curves for confined concrete. Evaluation of shear strength should be carried out following the 2002 AISC provisions or other codes applicable to the project.

Unlike reinforced concrete elements, nonlinear modelling parameters and inelastic deformation acceptance limits are not ready available for composite elements in FEMA 356 or ATC 40. Hence, these must be established either by laboratory tests or analysis (numerical virtual testing). In the latter case, moment-curvature analyses of composite sections using a program such as XTRACT need to be carried out subjected to a range of possible axial compression forces. This type of moment-curvature analysis establishes the effective cross section flexural stiffness value, $(EI)_{eff}$, and the ultimate (corresponding to the collapse prevention performance objective) curvature deformation capacity. The inelastic plastic hinge rotation angle acceptance limits for the life safety and the collapse prevention performance objectives can then be established following guidelines in FEMA 356 and ATC 40.

Deformation Criteria

Overall criteria for inter-story drift
Plastic deformation criteria for structural elements with some ductile capacity
Similar to FEMA 356 but with focus on high-rise structure types
Guidance on assessment for details not covered
Use of physical testing and 'virtual testing'

Strength requirements for elements with low ductility

Basis for provision of 'over strength' to non-ductile modes (e.g. shear vs flexure in RC core walls)
Treatment where shear and flexure strengths vary with axial stress during earthquake (e.g. RC core walls)

Energy dissipation components

Guidance on types and applications
Modelling
Specification and testing requirements

Non-structural elements

Assessment of deformation and strength demands
Detailing of connections
Movement joints