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CITIES WITHOUT A SEISMIC CODE II : CODIFICATION AND RISK ASSESSMENT

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SUMMARY

This paper which follows from the companion paper on "hazard modelling" addresses: (i) the development of design provisions for improving the protection of infrastructure against seismic risk for the future, and (ii) the assessment of seismic risk in existing infrastructure. A very important decision to make is the overall approach to be adopted in the codification. The provision of simple prescriptive detailing rules that waive the need for detailed analysis has been suggested. Alternatively, seismic protection could be incorporated by specifying nominal design forces. A contrasting approach is to stipulate elaborate analysis for irregular structures, which capitalizes on the capability of contemporary software in the handling of dynamic and non-linear problems. The merits and shortcomings of each of these approaches are examined. Debating openly in international forums like this conference will channel attention to the key issues in order that they can be addressed sufficiently early in the development of a new code or Standard. This paper also contains a critical appraisal of existing risk assessment methodologies.

INTRODUCTION

This paper, which follows from the companion paper entitled "Cities Without a Seismic Code I: Hazard Assessment" (Megawati *et al*, 2004) deals with the process of developing new regulatory documents for controlling future seismic risk in the built infrastructure. Numerous generic concepts which are central to codification for seismic design are first addressed. Contentious issues are addressed in a discussion format in revealing both sides of the argument. There is often no definite "right" or "wrong" to a dichotomy. The decision which strikes a good balance cannot be generalized as it is highly case dependent. Thus, this

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paper does not give conclusions to findings in a manner as done in reporting a scientific investigation. Debating openly in international forums like this conference will channel attention to the key issues in order that they can be addressed sufficiently early in the development of a new code or Standard. Specific provisions in which the authors have experienced difficulties have been short-listed to initiate debate. It is emphasized herein that efforts with codification must always be paralleled by similar efforts in the risk assessment of the existing building stock. The latter is addressed separately in a critical review at the end of the paper.

PLANNING FOR A NEW EARTHQUAKE STANDARD

In what Form, Scope and Approach ?

Seismic design provisions may be introduced in different forms. In the "all in one" form, every aspect of seismic design is introduced in one new document which covers the specification of both the seismic design forces and the design/detailing rules for every form of construction. The first seismic design Standard introduced in Australia in 1979 was of this form. At the time, this seemed to be an attractive option for a country that never previously had seismic design provisions in any existing Standard.

The committee/working group formed in developing the draft for the "all in one" Standard will need to address a wide range of issues covering engineering seismology, design load specifications, geotechnical matters as well as the design and detailing matters related to every major construction material including concrete, steel, masonry and timber. To limit the size of the committee to a manageable limit, no more than one representative from each discipline may be involved with the drafting in view of its very wide scope. Consequently, a consensus of opinion from experts on matters related to a specific material (eg. steel) could not be sought within the committee structure.

An opposite alternative to the "all in one" approach is the "no new Standard" approach, in which new detailing provisions in addressing seismic risks are incorporated directly into the respective material Standards through their regular revisions. The advantage with this approach is that no new Standard would need to be produced and consequently no new drafting committee is required (but experts on seismic matters must be invited to the respective material Standards committee). The impact on day-to-day engineering has been alleviated, as seismic provisions are introduced gradually with each constructional material. There are, however, shortcomings with this prescriptive approach of addressing seismic protection through detailing only, as discussed below.

The third approach, which is more commonly adopted by contemporary seismic design Standards, is de-coupling the codification for loading (or actions) from the codification for design/detailing. The advantage with this approach is the narrowing of the scope in drafting. The link between seismic actions and detailing in the conventional force-based methodology is through the specifications for the structural response factor (or ductility reduction factor), R_f. Different values for R_f can be specified for different classes of lateral load-resisting elements, depending on the level of ductility in the design/detailing. For example, limited ductile shear walls and fully ductile shear walls are assigned different R_f factors. It is important that detailing requirements for every class of element (defined by consistent terminology) have been incorporated into the respective material Standards by the time the "action" Standard is implemented. When this is not achievable, a lower bound, or default, value for R_f should be specified, in accordance with current practice.

Be Simple or Be Transparent?

Simplicity is always appealing to design professionals and has been a key consideration in the drafting of the code clauses. Transparency has also been an issue. Whilst simplicity and transparency are not necessarily incompatible, there are often situations where one is achieved at the expense of the other.

For example, a simple approach of specifying seismic load is by defining the magnitude of the loading as a percentage (e.g. 2%) of the seismic weight of the structure, irrespective of its natural period. This approach appeals particularly to designers with no prior knowledge of response spectrum procedures nor structural dynamic analyses. An alternative simple approach of introducing seismic protection is through detailing only (e.g. reducing the maximum limit to stirrup spacing or requiring lapped splices be staggered). The drawback with the first approach is its failure to address important trends in the actual seismic actions. Consequently, structures with higher natural periods are given more protection than those with lower natural periods. The second approach does not account for the failure mechanism of the structure. Both approaches are simple to apply but do not cultivate a good understanding of the underlying physical phenomenon. In this context, simplicity promotes a culture of "compliance" which is not synonymous with a culture of assuming genuine responsibility on seismic performance and the associated life-safety issues.

A culture of undertaking genuine responsibility can only be cultivated through engagement of the designer, who has a depth in understanding what really matters. A transparent Standard is thus desirable in this sense. Provisions involving the use of a design response spectrum procedure are more transparent than provisions based simply on detailing, since the former captures key characteristics of the dynamic response behaviour, by using the response spectrum. By similar arguments, step-by-step time-history analysis based on a representative hysteretic behaviour model of the structure is more transparent than simple static or elastic dynamic analysis involving the use of an empirical *ductility reduction factor* (or *structural response factor*). However, ambiguities associated with complex provisions could render the procedure difficult to implement in practice, as further discussed in the following section.

Clearly, the correct choice of a design approach should strike an optimal balance between the desire for both simplicity and transparency. The solution is dependent on the current state of practice and access to knowledge and expertise in the local professional community.

Issues with Ambiguities

In principle, it is expected that designers produce similar predictions for the seismic actions in the structure if parameters related to seismicity, site conditions and the structure itself have been well defined. Also, a rigorous procedure should provide more accurate predictions than a simplified procedure.

Whilst the above expectations appear sound and logical, reality is very different. For example, accelerograms required for time-history analyses may possess very uncertain individual characteristics, even when the modelling parameters have been well defined. Furthermore, the true behaviour of the structure is also subject to many uncertainties. In a recent comparative study, the computed stiffness properties for selected buildings of 10-20 storeys height, based on different (but acceptable and commonly used) modelling assumptions, were found to differ by up to a factor of 4. The corresponding natural period differs accordingly by a factor of 2 (Su *et al,* 2004). These uncertainties are strongly coupled, since the response behaviour of the building could be very sensitive to its natural period and damping. Consequently, different groups of designers working independently could produce very different predictions for the seismic actions in the structure.

In contrast, consistent solutions are likely to be produced by the different designers if static analysis or elastic response spectrum analysis has been used. Whilst such procedures (e.g. AS1170.4, 1993) might appear simplistic and lack the transparency (due to the use of empirical *R* factors), the design outcome is much more predictable due to the reduced ambiguities arising from a less complex procedure.

The same argument applies to the way in which natural period of a structure is calculated. Very simple algebraic expressions are available to define the natural period of the building as a function of its height, along with some broad structural classification. Alternatively, natural period could be computed from the analysis of a finite element (FE) model, or from the Rayleigh method (e,g. AS/NZS 1170.4-2002). These latter approaches may appear more accurate than using simple code expressions, but again, results obtained from the simpler approaches are more predictable despite being simplistic. It is debatable if the elaborate procedures necessarily provide more accurate predictions, given the sensitivity of the results to the modelling assumptions.

The use of a simple algebraic expression to define the natural period of the building appears to have circumvented the need to rely on the stiffnesses computed from the *FE* model. However, this is only the "first step" in the calculation. In the next step, the *FE* model is used in calculating the deflection and storey drifts based on seismic forces that have been calculated previously. Consequently, the stiffness assumed in the determination of the seismic forces (through simple code expressions) could be much higher than that used in calculating deflections and drifts. This important discrepancy, whilst not widely recognised, would result in the drifts and deflections being significantly overstated in the two-step calculation procedure of the *force-based* method.

The displacement response spectrum (as opposed to the usual acceleration response spectrum) has the attribute of determining deflections and drifts in a single-step (e.g Chandler *et al*, 2001). Errors associated with the use of incompatible stiffness, as explained above, are hence eliminated automatically.

Alternatively, the capacity spectrum procedure may be used in determining both the force and displacement demand at the same time from the acceleration-displacement response spectrum (ADRS) diagram (e,g. ATC40, 1996).

Both the displacement response spectrum procedures as introduced in the literature are iterative. The iterations are required to ascertain the appropriate amount of correction to the initial demand curves, which are based on 5% structural damping. Different schemes of applying the correction have been introduced (refer review by Miranda and Ruiz-Garcia, 2002), and consequently, there are potential ambiguities with these new procedures.

There are many areas in the seismic code that have similar problems with ambiguities. The calculation for displacement is only an example, used in the above illustration.

Is Incremental Change Better?

The above discussions point to the advantages in using the displacement response spectrum, or ADRS diagram, for the calculation of seismically induced displacements and drifts. However, these newly developed methodologies have only been evolved from international research for about a decade. The engineering profession in countries which have had little involvement with this research and development process may find them totally unfamiliar. Because of this, conventional force-based methods involving the use of R factors, still have a place in contemporary Standards. However, changes are clearly needed but must be introduced incrementally.

In a very recent draft of the new Australian Earthquake Loading Standard, the traditional R factor approach has been retained, as in the case of the new National Building Code (Canada) and the International Building Code (IBC, 2000). Significantly, a new provision which gives the user the option to calculate the *R* factor from push-over analysis has been introduced (BD006-04, 2004). It is stipulated that the *R* factor is defined as the ductility reduction factor, μ, divided by the structural performance factor, S_p (the latter could be interpreted as the reciprocal of the over-strength factor). Both the μ and the S_p factors could be inferred from the force-displacement (F-Δ) relationship developed from the push-over analysis of the structure. The attribute of this new provision is that the underlying meaning of the *R* factor is now made more transparent in the Standard. Furthermore, the F-Δ relationship is in the same form as the capacity curve (in the ADRS diagram) used in the capacity spectrum procedure (ATC40, 1996).

The engineering profession could be introduced subsequently to the idea of finding force and displacement demands by intercepting the capacity curve with the demand curves. This could be accomplished in the *Commentary* to the Standard or by separate publications addressing the new Standard. As commented earlier, there is a multitude of methods by which the initial demand curves for 5% damping could be corrected. Definitive and conservative recommendations would be needed to guide the profession, in order that potential ambiguities inherent in the new procedures are reduced as much as possible.

Two important points have arisen from the foregoing discussions. First, when drafting a new code or Standard, one must be very conscious of intrinsic drawbacks with existing methods. New innovations developed from recent research could then be assessed in terms of how effectively such drawbacks are addressed. Second, one must also be very cautious with new methodologies that have not been fully matured. Changes should be implemented incrementally so that the engineering profession could easily relate new concepts to existing ones. Ambiguities are always a cause for concern.

What are the Potential Impacts?

In the drafting of a new code or Standard, or new clauses in the revision to a Standard, the economical impact on the community as a result of implementation of the Standard must be analysed, to inform the decision makers.

First, the distribution of seismic resistant capacity in the existing building stock must be estimated. Such capacity estimates could be expressed in terms of the seismic coefficient (in units of g's) for each site class, that would result in the building not meeting the seismic performance criterion. For countries with no existing seismic loading provisions, reference could be made to existing robustness provisions or wind loading provisions in the assessment of the existing strength capacity. Structural response factors assumed in the strength capacity calculation must be justified with reference to local design and detailing practices, and not be taken by default from existing codes used elsewhere. The effects of soil amplification, including the possibility of developing high amplification pertaining to resonance conditions, must be taken into account in the assessment (Chandler *et al*, 2002, Lam *et al*, 2001).

With the average capacity of the existing buildings estimated, the proportion of the existing building stock that would not satisfy the proposed Standard requirements can be calculated for any given design return period and performance criterion. This assessment can be repeated for different return periods, to provide a clear perspective of the potential impacts of the Standard.

Further details on seismic risk assessment have been given at the end of this paper.

It has been argued that decisions on the design return period (which is based primarily on a consensus of opinion and is a function of the level of importance of the facility) are more than an "engineering decision". In Australia, specification for the design return period and importance classification for buildings is not contained within the loading Standard itself, but specified in a separate document by the *Building Code Authority*.

It is noted that the cost of the structural components in a typical building is less than half of its total construction cost. Furthermore, the construction cost is only part of its total operational cost. The economical impact of a Standard is more than a function of the strength requirements. For example, the introduction of new restrictions to certain form of construction (e.g. soft-storey, transfer structures, unreinforced masonry construction) would interfere with the architecture and hence the functionality of the building. Imposing such restrictions deserve careful considerations due to their far-reaching implications.

SPECIFIC COMMENTS ON THE DRAFTING

The production of a good Standard will involve careful planning of the form, scope, and approach by the committee in the pre-draft stage and painstakingly going through the draft clause-by-clause, following the production of the first draft. These objectives could only be fulfilled by a committee of experienced professionals and academics who would work constructively and harmoniously to represent a good balance of interests across the spectrum of the profession. Although the initial drafting could be out-sourced by contracts, substantial committee input is essential in the rest of the development process. This requirement is not exempted in situations where the first draft is based on an existing Standard used elsewhere.

The impact of a code or Standard on the workload of engineers and current design practices are as important an issue as the economical impact on the structure. Proper "Road Testing" of draft clauses by design professionals during the development of the Standard is desirable. A committee dominated by one or two “stars” is clearly not desirable. However, progress with the drafting is often hampered by working with a large committee (often well exceeding 10 members).

Below is a list of items that warrant special attention when drafting the clauses:

- Is it always easy to distinguish *regular* and *irregular*; *ductile* and *non-ductile* construction in practice? Should they be the criteria in dictating the type of analysis required for the building?

- Should minimum seismic design forces be expressed simply as a percentage of the seismic weight?

- Should dynamic analysis be made mandatory for important structures and high hazard sites; for irregular buildings; and for buildings exceeding a certain period range (height range)?

- What is the most effective way to allow for loading applied concurrently in two directions?

- To what extent vertical accelerations be allowed for in design? What are the justifications for such provisions?

- Should ductility factors be defined within the loading Standard or the respective material Standard?

- How should links be established between the seismic loading Standard and the materials Standards?

- Should the *R*-factor be dependent on the period of the building and that of the site?

- How should "Parts" (or "Non-structural" components) in a building be defined?

- Are the very high forces imposed on certain *NS* components (such as walls and partitions) justified? It is noted that very high forces have been stipulated at the roof of low-rise buildings.

- Is it appropriate to use *P-delta* effect provisions based on static conditions in a seismic loading Standard?

SEISMIC RISK ASSESSMENT

Seismic risk assessment for existing infrastructure and the development of codification for design are very much an integral part of the same process in mitigating and controlling seismic risks. In principle, the assessment should enable the potential scale and extent of damage, casualties and loss of life to be estimated, for earthquakes affecting the region. Importantly, as commented in the previous section, information provided by the feedback from the assessment is central to understanding the cost and benefit of introducing a new Standard and in deciding on the design hazard level in the Standard. However, the evaluation of risk posed by earthquakes to the structural safety of buildings in cities without an existing seismic code presents a number of challenges.

The Risk Assessment Methodology has no pre-defined international standard. The best known of existing approaches is the earthquake loss estimation methodology developed in the past seven years in the United States (US) by the Federal Emergency Management Agency (FEMA). The methodology forms the basis of the HAZUS loss estimation software (FEMA, 2000). It combines three basic sets of input, the first to represent the seismic ground motion (hazard spectra), the second to represent the response of buildings (capacity spectra), and the third to represent the vulnerability of buildings and its variability (fragility curves). Focus is placed here on the formulation of appropriate capacity spectra and fragility curves, which together present the greatest challenge in regions that have not considered seismic issues in design.

Capacity spectra (conventionally presented in acceleration-displacement format) describe how a “typical” building will behave under an increasing level of seismic loading, starting from its elastic behaviour and including post-elastic (non-linear) behaviour as the building approaches failure. Such curves are derived for a variety of building structural types and height ranges. The decision of building structural types and height ranges should clearly reflect those types that are prevalent in the region in question. This in itself presents a challenge to the risk analyst, as the default structural types and height ranges in HAZUS are often inadequate (or inappropriate) for use outside the US. For example, in Hong Kong, buildings below 10 storeys are considered low-rise whereas in the US, buildings of 8 storeys and above are considered to be high-rise. Such incompatibility must be addressed also in analyzing capacity curves to represent the building response. Capacity curves are usually obtained by a non-linear static push-over analysis, but this approach has limited viability for buildings in which higher-mode effects have a significant influence on the dynamic earthquake response; typically this applies for any building above 15 storeys. The implication is that the capacity of buildings above 15 storeys should be determined by a dynamic non-linear analysis, but this is neither economically viable nor technically feasible if the intention is to cover a range of building types and heights in a comprehensive manner.

To overcome some of the above problems, the so-called “typical” building types may be chosen to represent buildings within a given broad category of structural form and height range. The selection procedures for identifying such typical buildings have not been prescribed in HAZUS or other seismic risk assessment methods, and furthermore the extent to which they can represent the overall building population is questionable. Often the approach taken would be to identify a series of example buildings that are of a type commonly found in the region, such as tall apartment blocks or low-rise government schools. But the subjective judgement that is needed to select such examples remains a problem due to the lack of guidelines or even a set of criteria that may be used to guide the selection. Also, for practical reasons the number of such typical building types clearly has to be limited to perhaps 20-25, and whether that is sufficient for complex mega-cities with a wide variety of construction forms, types and ages is difficult to ascertain.

The second key component of building response within a seismic risk assessment methodology such as HAZUS is the development of vulnerability or fragility curves. The purpose of such curves is to describe the build-up of damage (to pre-defined levels, such as “minor” or “heavy”, which themselves are highly subjective terms) with the systematic increase of structural displacement or drift response. Inherent in the procedure is that the variability of structural capacity (such as at yield or ultimate conditions) is accounted for by the shape of the fragility curves. The shape is usually defaulted using the HAZUS assumptions of a log-normal distribution about the median level, and with a pre-defined level of variability defined in terms of the standard deviation of the distribution. The probability distribution is meant to reflect the existence of deviations from the assumed “typical” structural forms, in terms of building irregularity in its multitude of forms, the variable response of materials, and other such factors. It seems highly questionable whether the default parameters supplied in HAZUS are sufficiently robust for global applications, and yet seismic risk assessment studies may fall back on such parameters in the absence of any clear approach to account for local conditions.

A possible solution to the need to gain local information on building vulnerability would be to conduct detailed building inventories for selected districts within the study region, in which different forms of irregularity, building construction age and quality, materials used, and so forth, can be recorded and analysed statistically (e.g. Dimitrakoudi and Penelis, 2002). Without this information, only the default HAZUS parameters can be used. The sensitivity of the outcomes of the risk analysis to such assumptions has rarely been considered and there is an urgent need to ascertain to what extent the results are affected by the decision-making on the part of the analyst.

Finally, the risk analysis should be able to consider the potential impact of damage to non-structural components as well as structural elements. HAZUS provides separate, but compatible approaches to investigate the damage to each type of component, separating the principal structural elements, the drift-sensitive non-structural components and the acceleration-sensitive non-structural components. Once again, the ability of the default HAZUS parameters to be transported to other parts of the world outside the US has seemingly not been rigorously tested, and in some notable instances it has been assumed that the fragility curves developed for structural components may be equally valid for all types of non-structural component. This assumption appears intuitively flawed, as there is no *a priori* reason why non-structural components should have damage characteristics that are governed solely by the response of the main structure. The importance of accurate modelling of non-structural components in the risk analysis is evident when it is considered that in some parts of the world, such components account for up to 80% of the entire construction cost of a building.

It is evident from the above discussion that contemporary seismic risk evaluation methodologies such as HAZUS are, at best, capable only of providing a general indication of the potential damage scenarios in future earthquakes, in order that informed decisions on codification can be made. This would be the case, provided that all the modelling parameters have been calibrated to account for local construction practices. Even then, the assessment should not be relied upon to decide if individual structures are safe/unsafe or in making decisions on retrofitting. Such decisions could only be made following case-specific evaluation of a structure, and this is not within the capability of generic assessment tools such as HAZUS.

SUMMARY AND CONCLUDING REMARKS

- The three well known approaches, namely the “all in one”, “no new Standard” and the “seismic action only” approaches in codifying seismic design provisions have been introduced with reference to their attributes and shortcomings.
- The dichotomy of “simplicity” versus “transparency” has also been discussed along with the issue of ambiguities.
- When changes are desirable, their implementation should be incremental.
- Importantly, careful evaluation of the impact of changes brought about by codification is required for decision-making.
- Specific provisions in which the authors have experienced difficulties have been short-listed to initiate debate.
- Important limitations associated with existing risk assessment methodologies have also been highlighted.

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REFERENCES

1. AS/NZS 1170.4 -2002 : *Structural Design Actions : Part 4 : Earthquake Actions*, July 2003, Draft BD-006-270-VA1.
2. AS1170.4 - 1993. Standards Association of Australia. *Minimum design loads on structures* : Part 4: Earthquake Loads - AS1170.4 and Commentary.
3. AS2121-1979: Standards Association of Australia . *SAA Earthquake Code*.
4. ATC-40. Seismic evaluation and retrofit of concrete buildings, Vols.1&2. Redwood City, CA: Applied Technology Council, 1996.
5. BD006-04 (2004). Discussion at Meeting of Committee BD006-04 of Standards Australia for the drafting of the new Australian Earthquake Standard for Seismic Actions.
6. Chandler AM, Lam NTK and Sheikh MN. Response spectrum predictions for potential near-field and far-field earthquakes affecting Hong Kong: soil sites. *Soil Dynamics and Earthquake Engineering*, 2002; Vol.22, pp.419-440.
7. Chandler AM, Lam NTK, Wilson JL and Hutchinson GL. Review of Modern Concepts in the Engineering Interpretation of Earthquake Response Spectra, ICE Proceedings: Structures and Buildings, 2001.146(10), pp.75-84.
8. Dimitrakoudi KD, Penelis GG. Validation of the rapid visual screening of buildings for potential seismic hazard. 12th European Conference on Earthquake Engineering, Paper Ref. 027, 2002.

9. FEMA (2000). HAZUS 99 User's Manual, Earthquake Loss Estimation Methodology. Developed by the Federal Emergency Management Agency through a cooperative agreement with the National Institute of Building Sciences, Washington D.C., USA.
10. IBC (2000). International Code Council. International Building Code, USA.
11. Lam NTK, Wilson JL, Chandler AM. Seismic Displacement Response Spectrum Estimated from the Frame Analogy Soil Amplification Model, Journal of Engineering Structures, 2001; Vol.23: 1437-1452.
12. Megawati K, Lam NTK, Chandler AM and Pan T-C. "Cities without a seismic code I: hazard assessment." Proceedings of the 13th World Conference on Earthquake Engineering, Vancouver, Canada. Paper No. 129 (CD-ROM).
13. Miranda, E. and Ruiz-Garcia, J. "Evaluation of approximate methods to estimate maximum inelastic displacement demands", Journal of Earthquake Engineering & Structural Dynamics 2002; Vol.31, pp.539-560.
14. Su RKL, Chandler AM, Sheikh MN and Lam NTK. "Influence Of Non-Structural Components On Lateral Stiffness Of Tall Buildings", Journal of Structural Design of Tall Buildings, 2004 (submitted).