CANADIAN SEISMIC CODE PROVISIONS BEYOND 1985

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ABSTRACT

This paper outlines directions along which future Canadian seismic code changes beyond those made in NBCC 1985 should follow. Among the directions considered are changes in the base shear formula to reflect more directly how each factor influences the seismic loading. A proposal is made to classify building systems according to their expected seismic performance at ultimate limit state in which the reduction in base shear is tied to the expected performance classification. The level of sophistication in analysis, design, and detailing of construction should be compatible with the expected seismic performance level. It is suggested that guidelines provided by different material design codes would be sources of reference to achieve this aim. The idea of repairable damage limit state is discussed and some guidelines to accomplish damage control would be a very desirable feature in future Canadian codes.

INTRODUCTION

There has been considerable progress made in the fields of strong motion seismology and earthquake engineering during the last ten years. Such progress has resulted in improvement of seismic risk estimation, better understanding of soil and foundation effect on structural response, and methodologies and procedures of analysis and design to improve seismic resistance of structures in general. In Canada these new advances can and will be translated into design practice through different material design codes and the National Building Code of Canada (NBCC). In particular, the Canadian National Committee on Earthquake Engineering (CANCEE) is responsible for developing the seismic loading provisions in NBCC. Seismic code provision was first incorporated in NBCC in 1965 and changes have been in 1970, 1975, 1977 and 1980 editions of NBCC. The changes planned for NBCC 1985, which are described in a companion paper [1], represent a significant effort by CANCEE to incorporate the major advances in earthquake engineering of the last decade to the code format seismic provisions.

Although major code changes have appeared as a series of discrete changes over the years, the translation of research findings into practice is in fact a continuous process. Therefore, it would be useful to develop a document in the form of a seismic design guide by pooling all available technical information in a rational manner and continuously up-dating as changes are deemed necessary. This document would form the technical base for future code changes. Each proposed change could be evaluated on social and economic ground in addition to technical merit before the actual change is made to the building code. In this respect, such a seismic design guide is not dissimilar to the tentative seismic code provisions developed by the Applied Technology Council for the United States [2].

The purpose of this paper is to present some of the significant directions in which improvements could be made to the Canadian code to provide more consistent protection and improve design of buildings located in a seismic environment. The contents are also intended to be used as input in the formulation of a Canadian seismic design guide. Although the authors are members of CANCEE, the views expressed in this paper are solely those of the authors and are not intended to represent the views of CANCEE, nor of the Associate Committee on the National Building Code.

For convenience, the paper subdivides its contents under the headings relating to loading, analysis and design. Also, it is assumed that the proposed changes for NBCC 1985 will be adopted and the discussion will focus on the desirable changes beyond NBCC 1985.

LOADING

The seismic loading in building design begins with the calculation of base shear. The base shear formula planned for NBCC 1985 is [1]

$$V = vSKIFW \tag{1}$$

in which v = zonal velocity ratio
S = seismic response factor
K = structural behaviour factor
I = importance factor
F = foundation factor
and W = dead load.

While the base shear values obtained using this formula are believed to be reasonable, some changes are desirable to clarify for the user the physical basis for the different factors involved. The first change relates to the interpretation of the base shear V as computed. Currently, it is considered as a "service" load. The value of V calculated from equation (1) is to be multiplied by a load factor α_Q (= 1.5) in the design of buildings for strength. Since the structural behaviour coefficient (K factor) relates to the building behaviour at or near the ultimate limit state, it would be conceptually more consistent to calculate the ultimate load value within the seismic loading provisions, which would be used directly for strength design consideration with a load factor of 1.

The seismic response factor S in the base shear formula from NBCC 1985 consists of a period independent portion and a period dependent portion. The transitional period for these two portions is 0.25 s., the same as in NBCC 1980. In other words, assuming other factors remain the same, buildings with periods of 0.25 s or less need to be designed with a higher seismic coefficient than buildings with longer periods. However, many studies [3,4] based on recorded ground motions of western U.S. earthquakes show that statistically, the base shear coefficient is maximum for structures with periods around 0.4 s. Ground motions recorded on soft soil would produce maximum effects on structures with even longer periods [5]. This fact is reflected in the seismic response factors used in the seismic codes of many other countries [6,7]. Should the transition period for the S factor in the Canadian code be changed to 0.4 s? This is an important question that deserves serious consideration since there are many buildings built in seismic areas of Canada that fall within the period range of 0.25 to 0.4 s.

Another desirable change relates to the K factor in the formula. The inclusion of this factor recognizes the varying capability of different structural systems to dissipate energy during an earthquake and to realize and maintain ultimate strength at large inelastic deformations. Currently, the values of K range from 0.7 to 2.0 for buildings, with the lower values corresponding to systems which are recognized as being more ductile.

Even though the general variation of K (i.e., increasing K with decreasing ductility) is reasonable, there are two primary difficulties associated with the continued use of K in the present form. First, the K values do not carry any meaning in relation to building behaviour. For example, K = 1 does not imply elastic response for the building. Second, the different values of K are associated with specific building systems and are not defined in terms of expected building performance. It is therefore proposed to replace K by a reduction factor R in the denominator of the base shear formula. The value of R should be at or near unity for non-ductile structural systems for having responses to the design earthquake excitation and remain essentially elastic. The value of R would increase for structural systems with greater ductile deformation capacity.

However, if one simply replaces K by R without changing the description of the structural systems, there is no real improvement in the ability of the designer to use this information to design a building with a better capability to resist strong seismic motions. It is therefore suggested that the reduction factor R should be tied to the expected seismic performance of structural systems which are subjected to severe ground shaking rather than directly to different structural systems. One suggestion is to establish four categories of building systems A, B, C and D, ranked in decreasing order of ductility requirements as described in Table 1. The expected seismic performance in each category is defined and typical structural systems currently used are provided as examples. For instance, category A consists of any structural systems which are not only capable of developing their yield strengths, but are also capable of undergoing large inelastic deformation and dissipating a significant amount of energy through yielding during the earthquake without collapse. Typical examples of such systems are moment resisting frames, ductile tension-compression braced frames and ductile flexural coupled walls.

The advantages of this approach are twofold. First, it provides to the designer information on the important features expected of the building being designed. Second, classification according to expected seismic performance is less restrictive than the current table defining the K factors. Any new forms of construction or new structural systems can readily be classified into one of these categories provided that the seismic performance can be demonstrated. The value of the reduction factor R is then correlated to the seismic performance categories; i.e. the more ductile the system the larger the value of R. However, it is premature to suggest specific values of R for the categories in Table 1 at this time.

For important structures designed for essential public services, such as hospitals and fire stations etc., it is not sufficient that the buildings themselves are not damaged, but they should remain functional even after a major earthquake. In other words, sufficient drift control must be incorporated into the design of these buildings so that damage to nonstructural elements such as windows and partition walls can be avoided. In NBCC 1980 and continues in NBCC 1985, an importance factor I is incorporated in the base shear formula and a value equal to 1.3 is suggested for such important structures. The adequacy of this provision to ensure proper nonstructural element damage control is open to question. Currently, a load factor α_Q = 1.5 is used to change the "serviceability limit state" seismic load to the "ultimate limit state" seismic load, but the drift limitation is evaluated based on the "serviceability limit state" load. For important buildings which are required to remain functional after a severe earthquake, the drift limitation should be based on the "ultimate limit state" load to minimize non-structural damage. Consistent with this line of reasoning, the value of the importance factor I should at least be equal to that of the load factor, namely 1.5. It may be of interest to note that the values of the importance factor in the New Zealand and United States codes are 1.6 and 1.5, respectively [7,8].

The effect of soil on the seismic load is reflected in the soil factor F in the current code. Values of 1.0, 1.3 and 1.5 are prescribed for rock, firm soil and soft soil conditions. Some theoretical studies have indicated that the soil condition not only affects the magnitude, but also the shape of the response spectrum curve [5]. As a result, many seismic codes have incorporated the soil effect by prescribing different response spectrum curves for different soil types. The problem of foundation effect is a complex one and more study and field observations are necessary to guide the Canadian code development in this direction.

In summary, it is envisioned that the seismic loading in future code is given by a base shear formula in the form

$$V_u = \frac{v(SF)IW}{R} \tag{2}$$

The formula will be calibrated such that base shear V_u obtained in equation (2) represents the factored load, i.e. the load at the ultimate limit state. The numerator in equation (2) represents the base shear of the building responding elastically when subjected to the design based earthquake. It is this load value upon which drift calculations should be based on. The inelastic deformation capacity of the structural system is reflected in the reduction factor R. The grouping of the S and F factors indicates the possibility of using different shapes of seismic response factor to allow for the soil effects instead of using S and F as two independent factors as is currently done.

ANALYSIS

Since 1975, NBCC has allowed the use of dynamic analysis as an alternate route to obtain seismic loads for design. A procedure for dynamic analysis based on the response spectrum technique was given as Commentary K in the Supplement to NBCC. In 1977, a clause was added in the code to ensure that the dynamic base shear values used would not be less than 90% of the base shear obtained from the static code formula. Such a limitation is necessary because one may arrive at considerably lower base shear values using the dynamic procedure based on Commentary K. For NBCC 1985, CANCEE has recommended that dynamic analysis be removed as a specific option for determining base shear and its main use is to obtain the distribution of lateral seismic loads within the structure, particularly for irregular structures.

The difficulty in applying the dynamic procedure to obtain the base shear arises from the lack of calibration between the dynamic approach and the static approach. Even for regular uniform buildings, it has been shown [9] that the base shear determination based on the dynamic procedure can be substantially different from the static base shear formula, depending on the periods and structural types. Until some calibration is carried out in the future, the removal of dynamic analysis as a specific option for determining base shear is a reasonable interim measure to avoid the confusion caused by substantially different base shear values using the dynamic or the static approaches. However, to reconcile the base shears by the two approaches for regular structures will be a necessary task. This task is not an end in itself since it is generally recognized that the static approach provides a reasonable estimate of seismic loads on regular structures. The main purpose of making dynamic and static base shear results comparable for regular buildings is to provide calibration for the dynamic approach so that the designer may use this approach for the design of irregular structures with confidence.

Within the context of NBCC 1985, the recommendation is to use dynamic analysis for seismic load distribution purposes, particularly for irregular structures. However, defining classes of irregular structures for which the simple code formulae are not adequate and dynamic analysis should be used is a difficult, but necessary task.

Structures can be irregular in plan (horizontally irregular) or in elevation (vertically irregular) or both. A structure is irregular in plan when the loci of the centers of stiffness and centers of mass of the floors do not lie on two vertical axes. For multistorey buildings, irregular structures in plan imply that the eccentricity changes from floor to floor. The determination of the eccentricity in each floor can be a difficult task when different structural systems are involved. For example, there is no generally accepted procedure to determine the locations of the centers of stiffness, and hence eccentricities, for an asymmetrical wall-frame multi-storey building. Further study in this direction to provide guidance to designers to recognize irregular structures is necessary.

A structure is irregular in elevation when there is a drastic change in either mass or stiffness distribution along the height of the structure. The following sequence of steps may be used to confirm that the structure is irregular in elevation [10]: (a) Compute lateral forces and storey shears according to equivalent lateral load procedure. (b) Compute the lateral displacements of the structure as designed corresponding to lateral loading from step (a). (c) Compute new sets of lateral forces and storey shears by replacing h_x and h_i in the lateral load distribution formula found in NBCC by the displacements computed in step (b). (d) If at any storey, the computed storey shear in step (c) differs from the corresponding original value (from step (a)) by more than 25%, the structure can be considered irregular in elevation and modal analysis is a suggested alternative. If the difference is less than this value, modal analysis may not be required, and the structure should be designed using the storey shears obtained in step (c), since they represent an improvement over the results of step (a).

DESIGN

With the expected seismic performance of the buildings stated in Table 1, buildings should be designed and detailed such that their behaviour in the inelastic range fulfills the seismic performance expectation as defined in the Building System Categories. The procedure to achieve such performance differs for different building materials. The best source of information for designers is the appropriate material design code. It is therefore suggested that the material codes be encouraged to provide such information, if they have not already done so.

As far as the Canadian seismic code is concerned, statements should be provided in a commentary to guide designers to the appropriate sections of the material codes. For material codes with special provisions for seismic design such as the concrete code CAN-A23.3-M77 [11], specific sections of the material codes can be used. For material codes which do not currently have special provisions for seismic design, special comments can be given to be used as a general guide. The format for such information to be presented in a future seismic code could take the form shown in Appendix A.

Another important issue needed to be addressed in future code changes is the question of repairable damage. In contrast to the design for other loads, the objectives of earthquake resistant design are threefold, namely, to resist small earthquake disturbances with no damage, to resist moderate earthquakes without significant damage and to resist major earthquakes without collapse. The "no damage under small seismic disturbance" condition fits into the commonly accepted concept of the serviceability limit state and the ultimate limit state can be identified with the condition of severe damage but no collapse. However, the condition of repairable damage under moderate ground shaking is a seismic design objective which is not readily covered by the current serviceability limit state or ultimate limit state requirements, and is not considered explicitly in the current code.

Consider an illustrated example in which two buildings A and B are located in the same city and have the same natural periods. Building A uses a ductile structural system while building B uses a non-ductile structural system. In designing the strength of these buildings for a seismic disturbance of intensity level Q, the current code requires building B to be designed with a base shear equal to Q while building A would be designed with base shear equal to one-third of Q. The implication of such a requirement is that if an earthquake disturbance of intensity Q occurs, building B will remain elastic while building A will be damaged, but would have sufficient ductility and energy dissipation capacity to remain standing after the earthquake. However, if a moderate earthquake occurs and both buildings are exposed to an earthquake disturbance with an intensity equal to half of the design intensity Q, building B will remain elastic but building A will be damaged again, although not to the same extent as before. There is no indication that under this situation of moderate earthquake loading that the damage suffered by building A would be repairable (economically).

The problem of providing guidelines to cover the objective of repairable damage under moderate shaking is a complex task. One needs to quantify damage before one can define the threshold of non-repairable damage. One has to agree what is meant by moderate level of shaking based on both seismological and socio-economic considerations. But unless buildings are designed with the repairable damage condition in mind, the cost of repair can be so great that the buildings would be considered a total loss, in the economic sense, after a moderate earthquake. The concept of damage control needs to be specified in the code to alert the designer that it is another limit state which must be included in seismic design.

CONCLUSIONS

In this paper, a number of ideas are presented. Some are straight forward while others demand more critical study and evaluation. There is much work to be done to develop these ideas into code format. It is hoped that the materials presented will be useful input in considering future Canadian seismic code changes, and that these changes will lead to substantial improvement and simplification of the seismic design of buildings in Canada.

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TABLE 1

Building System Classification

Category	Expected Seismic Performance	Example Structural System
A	Subjected to design base earthquake, these buildings are expected to have undergone large inelastic deformation without collapse. Regions of plastic hinging are expected to develop throughout the building to provide effective energy dissipation mechanism. Therefore, careful proportioning and detailing of members are essential to produce good overall ductile behaviour as required.	Ductile moment-resisting frames. Ductile flexural coupled walls. Ductile tension-compression braced frames.
B	Subjected to design base earthquake, these buildings are expected to have gone into the inelastic range with some inelastic deformation experienced at plastic zone without collapse. Limited amount of energy dissipation from localized yielding is anticipated. Therefore, the building should be designed capable of developing its full yieldstrength with some degree of deformability incorporated in the yield regions.	Ductile wall system; Dual structural systems in which at least one system is capable of ductile behaviour such as ductile frames or ductile walls.
C	Subjected to design base earthquake, these buildings are expected to be stressed beyond the elastic limit at critical locations. Therefore, these buildings should be detailed at these critical locations to allow yielding to take place.	Cast-in-place, reinforced concrete or reinforced masonry construction. Tension-diagonal braced steel frame.
D	Buildings are expected to remain in the elastic range. There is no special requirement for buildings to exhibit structural strength beyond the elastic limit.	Unreinforced masonry, precast systems.

APPENDIX A

Design and Detail Requirement

As a general guide, buildings designed for different seismic performance category should be designed and detailed according to the following table.

TABLE A1

Category	Concrete	Steel	Masonry	Wood
A	CSA A23.3 Chapters 1-19		-	-
B	Same as A	(See Special Guideline)	-	-
C	CSA A23.3, Chapters 1-18 plus part of Chapter 19		-	-
D	CSA A23.3 Chapters 1-18		-	-

Special Guidelines for Steel Structure

Structural steel members and connections usually have inherent ductility, and if detailed according to the requirements of CAN3-S16.1-M78 [12], or CSA-S16-1969 [13] will have adequate member ductilities for buildings in Categories C and D and for many components of buildings in Categories A and B.

However, in order to achieve sufficient ductility so that the seismic performance expected of Categories A and B will be achieved, further consideration of details may be necessary. For the highest degree of ductile performance to be attained, details should correspond to those required for structures analysed plastically (i.e., plastic design sections, and sufficient lateral bracing near plastic hinge locations). Consideration should also be given to the following:

i) High localized strains should be avoided in regions of contained yielding. For example, bracing members and flange plates in moment connections should develop yielding of the gross section prior to developing the ultimate strength at the net section.

ii) Calculated forces arising from seismic actions may be exceeded, and connections in particular should be designed with the recognition that the members they connect may deliver larger forces than those calculated. Connections may therefore have to be designed for additional forces, up to and slightly exceeding the capacity of the weaker of the members they connect. Column splices, and beam-to-column connections in moment resisting frames are connections which may have to be treated in this way.

iii) Some details have been found to perform better than others when subjected to severe cyclic loading, and are therefore to be preferred. Examples include full penetration groove welded flange connections in beam-to-column moment resisting connections which are preferable to flange plates, and compression bracing members comprising a single integral section which are preferable to built-up bracing members.

A PRACTICAL APPROACH FOR PROBABILISTIC EVALUATION OF EARTHQUAKE SITE STABILITY APPLIED TO A CELLULAR WHARF BULKHEAD SYSTEM

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ABSTRACT

A generally applicable methodology for probabilistic evaluation of site liquefaction and submarine slope earthquake stability hazards, originally developed for application to a cellular wharf system, is formulated using available procedures based on (updatable) observations of site liquefaction and slope performance in previous earthquakes; the methodology is illustratively applied to the cellular wharf system. Pore pressure buildup, strength degradation, and associated slope stability effects are modeled. The probabilistic formulation accounts for uncertainty in site SPT characteristics and earthquake acceleration recurrence intervals. Numerical results (obtained without need of a computer) include probability estimates of stable site performance based on selected (1) minimum conventional factors of safety against liquefaction and (2) maximum calculated slope displacements, and using geotechnical data typically available on moderate, or larger, scale engineering projects. The methodology can be used to help clarify hazard risk levels and establish a sound basis for recommendation/ selection of project earthquake design details.

INTRODUCTION

The purpose of this paper is to present a general, practical approach for a probabilistic evaluation of earthquake site stability for shoreline sites located on cohesionless soils with nearby submarine slopes. The approach is documented and illustrated by its application to the State of Alaska's proposed Ferry Vessel Maintenance Facility at Ketchikan in southeastern Alaska (10); simplified geometric details of the project, including the cellular wharf bulkhead system, are shown in Fig 1. However, it is intended that the generality of the approach (for potential application to other projects) not be limited by the emphasis on Ketchikan project-specific data and assumptions, used here to demonstrate the approach.

Earthquake site stability as used here includes earthquake slope stability and earthquake-induced liquefaction in level ground. For important facilities adequate evaluation of the risk of stability loss by these hazards and determination of their acceptability in terms of economics and human safety is a necessary task; one which can be relatively uncertain and fuzzy.

The typical approach to stability is deterministic--using a minimum acceptable factor of safety. However, earthquake slope stability and liquefaction is often better formulated in a probabilistic manner. This provides a basis in the design process for quantifying risk to