EARTHQUAKE CODES AND DESIGN IN CANADA

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SYNOPSIS

The purpose of this paper is to discuss some particular aspects of seismic codes and design in order to clarify some of the issues and thereby assist the designer in applying aseismic design principles. It begins by discussing the purpose of seismic codes, with particular emphasis on codes applying to building structures. In this section, the objectives of the seismic loading provisions of the National Building Code of Canada (NBCC) are discussed in detail. This is followed by a short presentation of the seismic design philosophy in CANDU nuclear power plant design. A small study comparing wind and earthquake risk is described, concluding that the risks of failure are comparable. This is followed by a presentation of the relationship between response and excitation acceleration level for various ductilities which leads into a discussion of the role of serviceability in seismic design philosophy. Detailed discussions of the roles of dynamic analysis and ductility in seismic design are then presented, based on the current provisions of the NBCC. The paper concludes with a short discussion of the development of the seismic loading provisions of NBCC, including both historical perspectives and current developments.

RESUME

Cette communication a pour but de discuter quelques aspects particuliers des codes et calculs sismiques afin de clarifier quelques unes des conclusions et ainsi aider le calculateur dans l'application des principes de calcul antisismique. On discute d'abord du but des codes sismiques en portant une emphase particulière à ceuxqui s'appliquent aux bâtiments. Les objectifs des clauses de charge sismique du Code National du Bâtiment du Canada (CNBC) sont discutés en détails. C'est suivi d'une courte présentation sur la philosophie du calcul sismique du réacteur nucléaire CANDU. Une brève étude comparant les dangers de vent et de séisme est décrite et conclut que les dangers de rupture sont comparables. Ensuite on montre la relation entre comportement et niveau d'accélération de l'excitation pour différentes ductilités, ce qui conduit à la discussion du rôle des conditions de service dans la philosophie du calcul sismique. Des discussions détaillées des rôles de l'analyse dynamique et de la ductilité sont alors présentées, en se basant sur les clauses récentes du CNBC. On conclut avec un bref examen du développement des clauses de charge sismique du CNBC, incluant les perspectives historiques et les récentes techniques.

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INTRODUCTION

Canada has had various forms of seismic code in use for nearly four decades (1), primarily applicable to building construction. Such codes have had a major effect on design practice, particularly in the parts of the country located in the highest risk zone. During recent years there have been a number of significant code changes and these changes have inevitably raised many questions concerning the validity of the code and its applicability in the protection of structures.

The purpose of this paper is to discuss some particular aspects of seismic codes and design in order to clarify some of the issues and thereby assist the designer in applying aseismic design principles. The paper begins by a general discussion of the purposes of earthquake codes, with references to both Canadian and foreign developments. This will be followed by more particular discussions of earthquake loading in the overall design process. In addition, there will be specific discussion on the roles of dynamic analysis and ductility in seismic design. The paper concludes with a short discussion of recent and current developments within the National Building Code of Canada.

PURPOSE OF SEISMIC CODES

General

Seismic codes are essentially sets of rules devised for the design and construction of engineered facilities. These rules are intended to provide two basic objectives (2):

a) an acceptable degree of protection against injury and property damage due to the effects of the moderate earthquakes which may be expected to occur during the economic life of a structure, and

b) an acceptable assurance that lives are protected and structural collapse is prevented under the effects of a large catastrophic earthquake which might possibly (though quite improbably) occur during the life of the structure.

The foregoing objectives, even though stated very generally, do identify several valid areas of concern for a seismic code.

First, there is the concern for the protection of life, with the extent of protection related to the size of the earthquake, i.e. protection against injury during a moderate earthquake and against loss of life during a real catastrophic event. A second concern is for economic protection, again in relation to earthquake size, i.e. against damage to property during a moderate earthquake and against collapse during a major event. It is also important to point out the two levels of protection in both areas of concern. In general, there might very well be more than two levels, but it is the concept of ensuring a scale of protection which is of significance. Finally, there is the question of the acceptability of the level of protection or acceptable risk. This can be stated generally as a desire to reduce seismic risk to the same order as other risks which we accept within our society, but the particular expression of that desire does need to be quantified in some form. Later in this paper, the author will attempt to make an evaluation of the acceptability of the risk implicit in the seismic loading provision of the National Building Code of Canada by comparison with wind loading.

Building Codes

The building code which has been the forerunner of most seismic codes now in use in the world is that developed by the Structural Engineers Association of California, commonly known as the SEAOC Code (3). The Commentary of the 1974 version of the SEAOC Code indicates three objectives of earthquake resistant structural design:

a) Resist minor earthquakes without damage,

b) Resist moderate earthquakes without structural damage but with some nonstructural damage, and

c) Resist major earthquakes without collapse but with some structural as well as non-structural damage.

It can be seen that these objectives are similar to those stated earlier but differ in indicating a three level scale of protection against economic loss and in not explicitly including the concern for life protection. Protection of life is certainly implicit because of its general relationship to property damage protection, although one may speculate that the SEAOC Code does not specifically mention this aspect because it is written primarily to provide guidance to designers and not primarily to be used by building officials to ensure public safety.

A recent development is a document developed by the Applied Technology Council which contains tentative seismic design provision for use in the development of seismic code regulations for design and construction of buildings (4). This document, known as the ATC3 Provisions, is not strictly a code, although written in the same format. In the statement of philosophy, three objectives are included, the first two being identical with those of the SEAOC Code, and the third being similar but stated as:

> c. Resist major or severe earthquakes without major failure of the structural framework of the building or its component members and equipment, and to maintain life safety. It is also recognized that for certain critical facilities, particularly those essential to the public safety and well-being in case of emergency, criteria

> should be available to the designer which will permit design of a facility which will remain operational during and after an earthquake.

In addition, it is stated that the purpose of the ATC3 Provisions is "to establish design and construction criteria for buildings subject to earthquake motions in order to minimize the hazard to life and improve the capability of essential facilities to function during and after an earthquake".

It can be seen that life safety in the event of a severe earthquake has been stated explicitly in the objectives and that there is a general concern to minimize the hazard to life. There is also a recognition that critical facilities (e.g. medical facilities and power stations) need to remain operational during a major earthquake in order to permit the overall public safety to be ensured.

Since most other building codes applicable to modern urban societies have been developed from the SEAOC Code, the stated objectives are quite similar. Codes applicable to less developed areas have normally emphasized the protection of life, since the potential economic loss would not be very large.

National Building Code of Canada (NBCC)

The objectives of the earthquake-resistant design requirements of NBCC 1977 are described in Commentary J "Effects of Earthquakes" of that code (5). The primary objective is to provide minimum standards which assure an acceptable level of public safety by designing to prevent major failure and loss of life. Structures designed by its provisions should be able to resist moderate earthquakes without significant damage and resist major earthquakes without collapse, in which "collapse" is defined as that state which exists when exit of the occupants from the building has become impossible because of failure of the primary structure.

The objectives of NBCC 1977 are quite similar to those expressed in the ATC3 Provisions, with the exception that only a two level scale of protection is indicated. Since this paper's purpose is to discuss earthquake codes and design in Canada, a detailed discussion of the relationship between these objectives and the actual code provisions will be presented in the following paragraphs.

The first level of protection, i.e. resistance to moderate earthquakes without significant damage, implies that the primary structural framework has sufficient strength to prevent structural damage. The term "moderate earthquake" is not defined but could reasonably be assumed to be an earthquake which would result in structural response such that yielding is incipient in the primary framework. The strength control aspect of NBCC 1977 is primarily contained in the specification of a minimum lateral seismic force V. There are other provisions which describe how that force V is to be used in computing the various stress resultants, e.g. shear, overturning moments, and torsional moments. The force V is defined quite clearly for the various kinds of sub-systems, different seismic zones, different structural periods, different foundation conditions and different importance factors.

The second level of protection, i.e. resistance to major earthquakes without collapse, implies that the primary structural framework has sufficient ductility to allow extensive energy absorption by the structure without achieving a collapse state. Again the term "major" earthquake is undefined but would be considered to be any earthquake which would cause post-yielding behaviour in the primary structural framework. The ductility control aspect of NBCC 1977 is contained within the specification of the lateral force V, primarily in the specification of a numerical coefficient K which varies with the type of construction. A small value of K is assigned to the most ductile systems and K increases as the systems become less ductile in nature. This coefficient is in recognition that the acceleration response of yielding systems decreases as the amount of post-yielding deformation increases.

Even though the objectives of NBCC 1977 make no direct mention of concern for property damage, it would be expected that proper design would also ensure that non-structural damage during a moderate earthquake would not be extensive. This can be inferred because many of the provisions of NBCC 1977 have been derived from SEAOC, whose objectives include this aspect. Insofar as structural behaviour is concerned, this damage control is ensured by providing adequate stiffness in the structural framework, thus limiting lateral deflection and distortion within the building. The damage-control provisions of NBCC 1977 are contained in a small section (4.1.9.2) near the end of the seismic loading provisions. The sentences of this section include some general statements on: the need to consider lateral drift, the need to provide clearances for non-integral structural units, the need to consider load transfer to non-structural components and the need to prevent collision of adjacent buildings. It should be noted that all these needs are expressed in very general terms with no numerical or quantitative limits stated. For example, it is simply stated that lateral drift shall be considered in accordance with accepted practice. It is clear therefore that this damage-control level of protection is of quite secondary importance within NBCC 1977 and the actual level of protection is almost entirely dependent upon the designer's concerns (and presumably the owner's as well).

Other Codes

Seismic codes are not restricted to those governing the design and construction of buildings, but include such jurisdictions as nuclear power plants, bridges, and foundations. This section will present a short discussion of the objectives of seismic design in one of these other areas and its relationship to the building code.

CANDU nuclear power plants are thoroughly qualified to resist potential earthquakes, because of the great emphasis placed on nuclear safety. Up to this time, seismic design requirements were developed by mutual agreement between Atomic Energy of Canada Limited (AECL), the

Canadian licensing authority (Atomic Energy Control Board) and the client utility. However, the situation has become more complex in recent years, both because of increasing utilization within Canada and because of the export of CANDU systems and technology to other countries. An additional factor has been the increased public awareness and concern for the safety of nuclear power plants. This has resulted in considerable effort toward the development of an appropriate code by the CSA Technical Committee on Seismic Design, which has responsibility for developing a code entitled "Seismic Qualification of CANDU Nuclear Power Plants (CSA N289)". As of this date, this Committee has approved the first part of that code (CSA Standard N289.1, General Requirements for Seismic Qualification of CANDU Nuclear Power Plants), which contains the general objectives and philosophy (6). These can be summarized by the statement that the nuclear power plant is to be designed and constructed to ensure that the effects of an earthquake do not lead to unacceptable radiation exposure of the public.

This philosophy is implemented by defining two earthquake levels:

a) Design Basis Earthquake (DBE) is an artificial representation of the combined effects, at the site, of a set of possible earthquakes having a very small probability of exceedance during the life of the plant, and

b) Site Design Earthquake (SDE) is an artificial representation of the combined effects, at the site, of a set of possible earthquakes having an occurrence rate of 0.01 per year, based on historical records of actual earthquakes applicable to the site, with the provision that the peak ground acceleration shall not be less than 0.03g.

The standard specifies that the reactor should be shutdown safely, including removal of decay heat from fuel and radioactivity contained within the containment building in the event of a DBE level earthquake occurring. The SDE level is used primarily to ensure that the emergency core cooling system would remain functional in the event that such a level earthquake occurred within a short time after the unlikely event of an independent loss of coolant accident.

Consequently, it can be seen that the specific design requirements are entirely related to safety considerations and not to economic concerns. There is an additional requirement that the NBCC govern the design of all non-safety related structures and systems. This requirement ensures some minimum level of economic protection against damage. In addition, there is a clear statement that the Owner may specify, for reasons of economic concern, that non-safety structures and systems should also be designed to the more restrictive requirements of the CSA N289 code.

It is of some interest to consider the comparison of nuclear and non-nuclear building design requirements. Duff (7) has made such a comparison by considering the ground accelerations and seismic response ratios for the CANDU nuclear power plant being constructed at Point LePreau, New Brunswick. This comparison is summarized in Table 1.

From this table it can be seen that the DBE ground acceleration is 5 times that specified by NBCC*. The determination of response parameters was by elastic dynamic analysis for the DBE and by static analysis for NBCC. The factors in the second column of the table indicate the increased amplifications which result from using the dynamic analysis. These comparisons give some indication of the increased strength requirements for seismic design of nuclear power plants compared with normal building structures.

EARTHQUAKE DESIGN

Comparison of Earthquake and Wind Loading

Earlier in this paper it was mentioned that the risk of property damage or to loss of life should be acceptable in the sense of being comparable to other risks which are encountered. It is difficult to make a general risk evaluation but it is useful to make a particular comparison between seismic risk and wind risk, since both have some common features. From the point of view of NBCC 1977, such a comparison is particularly appropriate since wind and earthquake loads are considered as alternative governing lateral loads in the load combination expressions.

In order to make such a comparison on a general basis, several simplifying assumptions will be made. First, it is assumed that a valid comparison is based on comparing probabilities of reaching ultimate load, i.e. a comparison by strength. Second, it is assumed that the NBCC loads in each case (multiplied by the appropriate load factor) represent the real collapse loads.

Consider first the probability of reaching ultimate level seismic load. The design force V in NBCC 1977 is assumed to be proportional to the ground acceleration A_{100} having an average annual probability (of being exceeded) P = 0.01. However, the ultimate level force V will occur at a ground acceleration $A_U = \alpha_Q A_{100}$ in which α_Q is the applicable load factor, which is specified to be 1.5 in NBCC 1977. Using the data provided in Commentary J of NBCC 1977, Fig. 1 shows curves of A/A_{100} versus probability P for several locations within Canada. Interpolating on these curves yields collapse probabilities (at acceleration $A_U/A_{100} = 1.5$) varying from 0.0053 to 0.007.

The design lateral force due to wind for purposes of the strength aspect of design is proportional to the hourly mean wind pressure q_{30} having an average annual probability (of being exceeded) P = 0.033. The collapse level load, by the same rationale as described above, will occur at a wind pressure $q_u = 1.5\ q_{30}$. Davenport, based on

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*The value used as the NBCC acceleration in ref. (7) is actually A_{100} and not the zonal acceleration specified in NBCC 1977, which would be 0.04g rather than 0.03g.

experimental observations in the U.S. and in other parts of the world (8), has shown that wind velocities fit a Weibull probability distribution, in which velocity is related to probability by the following proportionality:

$$\nu \alpha [-\ell n\ P]^{1/k} \quad (1)$$

in which k is a parameter varying with geographical location, and having values predominantly in the range 1.7 to 2.0. Since pressure is proportional to the square of velocity, then

$$q \alpha [-\ell n\ P]^{2/k} \quad (2)$$

Using the proportionality of Eq. 2 above, curves of q/q_{30} are also shown in Fig. 1 for values of k = 1.7 and 2.0. Interpolating on these curves yields collapse probabilities at wind pressures q_u/q_{30} = 1.5 varying from 0.006 to 0.008.

By comparing the wind and earthquake collapse probabilities it can be seen that they cover approximately the same range. It can therefore be concluded that the structural collapse risks due to wind and earthquake are approximately the same. Consequently if wind risks are acceptable to society, then the level of earthquake expressed in NBCC 1977 should also be acceptable, bearing in mind the simplifying assumptions expressed previously.

Acceleration-Response Relationships

The purpose of this section is to present a simplified analysis of the relationship between the intensity of seismic excitation and the type of response expected in a particular type of structure. In order to do this in a reasonably general way, it is again necessary to identify several simplifying assumptions. First, it is assumed that a single measure of seismic intensity is the level of peak ground acceleration, which is consistent with the provisions of NBCC 1977. Second, it is assumed that the complex structural stiffness characteristics can be represented by a simple elastic perfectly plastic relationship between base shear V and some deformation parameter δ e.g. lateral deflection at the top of the structure. Relative to this stiffness relationship, it is convenient to define the following quantities:

V_y = yield level base shear

δ_y = yield deformation

δ_u = ultimate deformation

μ = δ_u/δ_y = ductility capacity

When this type of structural system deforms inelastically, it is assumed that the actual maximum acceleration response is obtained by dividing the elastic spectral value by the ductility capacity μ which is valid for structural periods $T \geq 0.5$ sec. (9, 10).

In terms of the behaviour of the structure, it is useful to identify levels of input acceleration corresponding to three levels of structural response:

A_u = acceleration which would produce ultimate or collapse deformation δ_u,

A_y = acceleration which would produce yield deformation δ_y, and

A_e = acceleration which would produce an elastic deformation $\delta_e < \delta_y$ which is set at a level such that non-structural damage would be negligible.

This third level has been identified because of the concern expressed previously for the damage control level of protection, i.e. a serviceability condition. Because of the linear elastic relationship below yield level, δ_e can be defined as $\delta_e = \beta\delta_y$ ($\beta<1$) and the acceleration levels are related by $A_e = \beta A_y$. The base shears and deformations for the various earthquake levels, expressed in terms of the yield level parameters, are tabulated in Table 2.

A further assumption is that the design shear V_d, as defined in NBCC 1977, is associated with some design level of ground acceleration A_d, which is not necessarily the ground acceleration parameter A defined in NBCC 1977. This last point should be particularly noted, since the values of the parameter A (and the associated response parameter S) in NBC 1977 were assigned to calibrate the base shear values to those of previous editions of the code (1). This aspect is discussed in more detail later in this paper.

By recognizing that $V_u = \alpha_Q V_d$ and that V_u will govern the collapse of the structure, Table 2 can be used to develop the relationship between structural response and input acceleration for varying ductility capacity, which is shown in Fig. 2, assuming $\beta = 0.60$ and $\alpha_Q = 1.5$. From this representation it can be seen that A_u/A_d does not vary with ductility factor, which is just a restatement of the condition that design is based on ultimate capacity. However, it is of considerable interest to see the effect of this on the behaviour at lower levels of acceleration. At acceleration levels $a/A_d \geq 1$, there will be post-yielding deformation for most structures, except those having very small ductility capacities. However, if the acceleration levels are relatively low, e.g. $a/A_d = 0.5$, then one can make some interesting observations. The structures having little ductility capacity will have little or no damage of any kind; those of intermediate ductility capacity will have considerable non-structural damage and those with high ductility capacity will suffer some structural damage because of post-yielding deformation.

It is recognized that this analysis has been somewhat simplified so that the actual numerical values cannot be directly used to draw inferences for specific structures in particular locations. However, the general implications should be quite clear, i.e. more ductile structures will suffer more damage during minor to moderate earthquakes than those having less ductility. This is a direct result of utilizing the inelastic capacity of ductile systems to reduce the design level forces. This is true in NBCC 1977 for both the static method in the code proper (because of using the K factor) and for the dynamic analysis method described in Commentary K (because of using the $1/\mu$ multiplying factor to obtain the inelastic spectral acceleration from the elastic spectrum). This may seem startling to some observers, but it is definitely consistent with the stated code objectives, which emphasize protection against collapse and give minimal mention of protection against property damage.

The above analysis is nothing more than an interpretation of the effects of current seismic design philosophy as expressed in NBCC 1977. In order to stimulate discussion of our Canadian seismic design philosophy, the author would like to question whether this philosophy is appropriate. Consider the following points. In the last 50 years, there has been on average one earthquake each decade in Eastern Canada with a magnitude greater than 6, and two each decade in Western Canada with magnitude greater than 6.5. Many of these have been offshore or in relatively unpopulated regions, e.g. the Canadian arctic. To the author's knowledge, no Canadian earthquake in the last 75 years has caused any loss of life. Given these conditions, is it realistic that seismic design in Canada be governed almost entirely by collapse prevention philosophy? To face the problem from another direction, there is reasonable evidence that the seismicity of the more populated parts of Canada is substantially lower than that of California (4). If that is the case, is it reasonable that Canadian design philosophy have essentially the same basis as parts of the world having much higher seismicities? This is not to suggest that the concern for protection against loss of life should be eliminated but that it might be realistic to have a more balanced set of design objectives.

The author has no specific recommendations to make on this matter at the present time. However there is one direction which would appear to be worth investigating. Perhaps the basic quantitative design should be associated with the minimization of damage and qualitative provisions be used to provide the protection against collapse. Such qualitative provisions might include consideration of construction quality, inherent energy absorption capability of particular forms of construction and the suitability of the structural layout. This direction would have additional merit in avoiding the necessity to use specific numerical values for ductility capacity, particularly since the evaluation of ductility capacity is an uncertain and tenuous matter, governed by many assumptions of doubtful validity.

ROLE OF DYNAMIC ANALYSIS IN DESIGN

Reasons for Dynamic Analysis

NBCC 1977 permits dynamic analysis as an alternative to the static procedure described in the actual code seismic loading provisions, and describes an acceptable form of dynamic analysis in Commentary K of that code. While permitting dynamic analysis, NBCC 1977 restricts the dynamically determined base shear V to a value not less than 90 percent of that determined by the static analysis of the code. There has been considerable confusion as to the circumstances in which a dynamic analysis would be necessary and it is therefore of some practical interest to discuss the reasons for doing a dynamic analysis.

Basically there are only two reasons why a dynamic analysis would be appropriate in the aseismic design of a building:

1. for structures which are irregular in mass and/or stiffness either in the vertical domain or in plan.

2. when the dynamic motion has the possibility of significantly amplifying the response over and above that incorporated in the static analysis procedure.

Considering the first of the above, it is useful to quickly review the nature of the static loading provisions with respect to the distribution of seismic forces along the height of the structure. Regardless of the magnitude, the total seismic lateral force V is distributed vertically in proportion to the mass times the height from the base, with the exception that a portion of the total load is applied as a concentrated load at the top for very slender structures. Basically this distribution is an attempt to simulate the maximum dynamically induced inertial forces. For structures of relatively uniform mass distribution, this is usually quite adequate but can be quite considerably in error for non-uniform mass distribution. The code distribution of forces does not allow variation with stiffness non-uniformity, which certainly would be the case in an actual structure. Some examples of vertical mass and stiffness irregularities which would suggest the need for a dynamic analysis are:

a. a relatively open and/or tall bottom storey, with higher lateral stiffnesses in the storeys above,

b. a building structure with one or more setbacks,

c. discontinuities of size and/or location of lateral stiffening elements, and

d. large masses attached to a non-uniform structural frame, as in certain industrial building structures.

Relative to irregularities in plan, either mass or stiffness, it is often assumed that the torsional moments computed in Paragraph 4.1.9.1(16) of NBCC 1977 are intended to fully take account of such irregularities. However this provision is primarily intended to provide for relatively uniform asymmetry with respect to height but is not meant to cover asymmetry which varies with height. Even for regular asymmetry, these provisions may not adequately describe the distribution of lateral forces due to torsional effects.

The 1974 edition of the SEAOC Code (3) contains the following statement which incorporates this first reason for dynamic analysis:

> "The distribution of the lateral forces in structures which have highly irregular shapes, large differences in lateral resistance or stiffness between adjacent stories or other unusual structural features shall be determined considering the dynamic characteristics of the structure."

The recently developed ATC3 Provisions have attempted to more clearly define the circumstances in which dynamic analysis is required, using the same reasoning as outlined above. It is useful to briefly summarize the approach taken. First all buildings are assigned to one of four seismic performance categories based on both an assessment of level of seismic risk (seismicity index) and the importance of the building (seismic hazard exposure). These categories are defined by letters A, B, C and D in order of increasing performance requirements. Second, each building is classified as regular or irregular in both the plan and vertical configurations, using specified criteria. Then, the minimum level of analysis for any building is specified, based on the performance category and whether the building is regular or irregular. Any building in categories A or B, whether regular or irregular, may be analysed by some type of static procedure, whereas irregular buildings in categories C or D shall be analysed by some form of dynamic analysis.

The second reason for dynamic analysis is exemplified by the phenomenon of modal coupling between lateral and torsional motions during the response of a structure. Even for "almost-symmetrical" buildings, a highly amplified torsional response can occur when the fundamental torsional and lateral periods are quite near to each other (11, 12). Fig. 3 shows the implications of this phenomenon for a simple uniform flexural cantilever structure, with the provisions of NBCC 1977 superimposed. This clearly indicates that the static code provisions are inadequate if the fundamental uncoupled* lateral and torsional periods are within 20 percent of each other. Detailed dynamic studies of building response (13) indicate that the torsional response may be amplified by a factor of 3 or more over that computed by a static analysis which already includes static eccentricity. At the present time the effects of modal coupling cannot be codified because of the complexities of the torsional-lateral interaction, but this phenomenon is "flagged" in a footnote to paragraph 4.1.9.1 (16)

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*Uncoupled periods are those computed assuming that the mass and stiffness centres coincide, thereby uncoupling the lateral and torsional response.

of NBCC 1977 as well as being discussed in Commentary K. The guideline at the present time is that a dynamic analysis should be conducted when the fundamental uncoupled torsional and lateral periods are within 20 percent of each other.

It is sometimes assumed that a dynamic analysis will produce a more accurate or valid value of base shear. As will be discussed in the next section, the assumptions involved in determining the magnitude of the dynamic forces produce at least the same level of uncertainty as the assumptions used in determining the static seismic base shear. Consequently, it is not true to consider that the dynamic value is a better estimate of what really happens. Therefore the objective of yielding a reduced base shear by a dynamic analysis is not valid.

Dynamic Analysis Methods and Assumptions

The dynamic analysis procedures actually used for the design of structures can be broadly classified into two methods, namely Modal Analysis (MA) and Time History Analysis (THA). The method described in Commentary K of NBCC 1977 is fairly general form of MA, which is the most commonly used method of dynamic analysis. It is important that the user of either method be aware and have some understanding of the underlying assumptions. These will be discussed in some detail for MA, followed by a shorter discussion of the THA method.

For MA, the first assumption is that the structure deforms in a linear elastic manner during its seismic response, thereby permitting each mode to be analysed separately, with the results being combined in some manner. Each mode is assumed to behave as a single degree of freedom system, with its own damping, natural frequency and mode shape. The maximum elastic response parameters in any mode are determined by using the ordinates of a seismic response spectrum, for the particular level of damping which has been assumed, at the frequency of that mode. For design purposes, a set of average design spectra are normally used, such as given in Appendix K of NBCC 1977.

This set of spectra, for different levels of damping, are reproduced in Fig. 4, for a peak ground acceleration of 1g; for any particular value of design ground acceleration A, the ordinates of Fig. 4 are scaled linearly. This particular set of spectra is based on a statistical study of the response spectra of a large number of recorded earthquakes (14). For any particular level of damping the spectrum consists of a set of straight line bounds which represent the amount by which the original ground motion is amplified. Since each spectrum is a so-called "average", it is important to interpret the meaning of this "average". For this particular set of spectra, approximately 16 percent of all earthquakes would cause structural response levels which exceed the bounds in Fig. 4, i.e. the bounds are established at a probability level of non-exceedance equivalent to the mean plus one standard deviation (for an assumed log-normal distribution). In order to have some understanding of the order of variability of structural response, the additional amplification to go from the true "mean" to the "mean plus one standard deviation" can be as high as 50 percent, with the actual value depending upon both the damping and the period. It is also true, even though no quantitative measure is possible, that a certain proportion of structures having different periods will have response amplifications in excess of those given by the average spectrum, when subjected to any particular earthquake.

As mentioned previously, when the maximum response in each mode has been determined, it is necessary to superimpose the modal maxima in some manner in order to obtain the overall maximum response. Direct absolute summation of maxima will clearly over-estimate the response by a substantial amount, simply because of the insignificant likelihood of all modal maxima occurring at the same instant of time. For multi-degree of freedom structures, it has been shown that the square root of the sum of the squares (SRSS) of the modal maxima yields the most probable maximum response, provided that the natural periods are well separated (15). For buildings responding with essentially in-plane motion, the SRSS procedure will yield quite reliable results. However, if two of the periods are quite near to each other (particularly the first and second), then there is considerable likelihood that the corresponding modal maxima will occur at the same time and it would be advisable to have a direct summation of those particular maxima. A practical guideline for what is meant by "near to each other", would be a 20 percent separation or less.

For buildings responding in combined torsion and lateral motion, there has been some concern as to the applicability of the SRSS procedure, even when the periods are well separated. However, it has recently been shown (16) that the procedure can provide reliable maxima provided that the lateral and torsional components of response in each coupled mode are treated together and not separately, as is often done in conventional practice.

The foregoing discussion has concerned itself entirely with the MA method. For THA, a specific time-history of ground acceleration is used as input to the structure, with numerical integration in the time-domain to obtain the time-history of all the significant response parameters. The maximum of each of these can then be determined and used for proportioning of members, etc. The major problem with the THA method is the selection of an appropriate ground motion. If an actual earthquake record is used, the response computations will be valid for that earthquake record only and will not be valid for any other record. It is possible to generate an artificial earthquake record which is compatible with a specific design spectrum, but the results will still only be valid for that particular record. It is always possible to use an ensemble of records but the cost of analysis could then become prohibitive.

For both the MA and THA methods, it is necessary to choose a value of peak ground acceleration to be used in specifying the input level. NBCC 1977 specifies that the input ground acceleration be not less than the zonal design ground acceleration A, as specified in the Table of Climatic Data (Supplement No. 1 to NBCC 1977). The zoning map, from which A is determined, is based on peak horizontal ground accelerations having an average probability of exceedance of 0.01 per annum. Consequently, unless a designer wishes to make a more detailed evaluation of seismic risk at a site, the input level is precisely the same as that used in the static load computation. The value of A is constant in each Zone, but engineers should be aware that within Zone 3, peak horizontal ground acceleration corresponding to 1 in 100 average probability of annual exceedance can be significantly larger than value of A.

An additional parameter common to both methods is that of damping, which has considerable influence on the level of response. Damping is a complex quantity which is impossible to determine except by direct experimental measurement. Even then, it is dependent upon both the nature of the induced response and on the amplitude of response. For simplicity, engineers use the concept of equivalent percentage of critical damping, which is a measure of the rate of decay of response when a structure is in free vibration. Zero percent implies no decay and 100 percent means pure decay and no oscillation. For structures responding elastically, the equivalent damping percentage can vary from 0.5 to 7; with some inelastic response these increase to a range of 2 to 15. Commentary K of NBCC 1975 gives a tabulation of three possible values for different types of structures. A more detailed compilation is given by Newmark and Hall (10), which includes effect of stress level. Nevertheless, it should be recognized that the choice of a particular level of damping adds another measure of uncertainty, both because of the features of the structure and the effect of level of response.

The foregoing discussion has considered only elastic dynamic analysis; the effects of inelastic behaviour on both design and response calculations will be discussed in a subsequent section of this paper. For an elastic structure, either the MA or THA method will yield fairly accurate distribution of the primary stress resultant parameter, e.g. inter-storey shear and overturning moment. Figs. 5 and 6 illustrate this by showing comparisons of those parameters for two different structures, analysed by both MA and by the static method of NBCC 1977. By normalizing relative to the base values, it is possible to compare only the distributions with height. For both the structure with relatively uniform mass distribution and that with a large concentrated mass at the top, both methods yield quite similar distributions. For more unusual mass and/or stiffness distributions, the distribution can differ considerably and in such instances the dynamically determined distribution should be used.

The most significant variations between dynamic analysis and the static approach occur in the magnitude of the stress resultants which are obtained. This is illustrated in Fig. 7, which is based on a study conducted by Tso (17). This figure shows a comparison of elastic base shear values obtained using MA and the static method of NBCC 1977, for both shear wall and frame structures, each having uniform mass distribution with height. The absolute value of the ratio shown will vary according to the choice of damping, and/or K value, but the shapes of the curves will remain essentially the same. For example, if 5 percent damping is used rather than 2 percent, the ratios would all decrease by approximately 20 percent.

Considering first the effect of natural period, it can be seen that large variations occur for both kinds of structures, although the shear wall variation is near to being constant for periods greater than 1 second. This variation is due to the simple manner in which the static S factor is NBCC 1977 attempts to simulate the response of the structure in its various modes.

Considering the two types of structures, several significant differences can be observed. Firstly, it can be seen that the higher modes are much more significant for shear walls than for frames. This is due to substantial differences in ratios of second period to fundamental period and to differences in the modal participation factors for the two types of structures. Secondly, it can be seen that for taller structures (i.e. higher periods) frames have substantially lower base shears than shear walls. This is again due to the large higher mode participation in shear wall response, since the fundamental mode values are not too different.

The foregoing discussion clearly demonstrates that dynamic analysis should be used primarily to obtain more accurate distribution of stress resultants for non-uniform structures. In such cases, the absolute values of these stress resultants are subject to a great deal of uncertainty due to the various assumptions which must be made. Consequently, there is justification for limiting the dynamically determined base shear to a minimum of 90 percent of that determined by the static base shear, as specified in NBCC 1977. The ATC3 Provisions also limit the reduction of base shear that can be achieved by MA compared to the use of the static equivalent lateral force procedure. Such a reduction, where it occurs, is thought justified because of the more accurate representation of earthquake response given by MA.

ROLE OF DUCTILITY IN DESIGN

From the previous discussion in this paper, it is clear that ductility has an extremely significant role in seismic design. First, all seismic codes require that structures be designed to resist severe earthquakes without major failure or collapse of the primary load-resisting system. In practice, this can only be achieved if a structure has sufficient energy-dissipating capability to be able to allow the structure to deform and still retain its integrity. Second, the presence of ductility capacity in a structure has the effect of decreasing acceleration response even though large deformations are occurring, albeit in a complex manner. It is the purpose of this section of the paper to consider three aspects of ductility in seismic design:

a) effect on design forces,

b) effect on dynamic response, and

c) ductility capacity of structural system.

Effect of Ductility on Design Forces

For the static method of determining the design forces as defined in NBCC 1977, the effect of ductility is included in the coefficient K; Table 3 gives abbreviated definitions of the types of buildings associated with different values of K. Similar definitions are prescribed in the SEAOC Code. This coefficient is one of the multipliers in determining the lateral seismic force V. It can be seen that there is clear hierarchy of increasing value of K with decreasing overall ductility, even though ductility capacity is not measured in any real quantitative manner. The ATC3 Provisions contain a response modification factor R in the denominator of the expression used for calculating V; this modifier is large for more ductile systems and small for the more brittle systems.

When modal analysis is used, as exemplified in Commentary K of NBCC 1977, the most common procedure is to estimate the maximum structural ductility capacity μ so that for a given value of μ, the maximum inelastic spectral accelerations and displacements can be determined by modifying the elastic average design spectrum. For modal periods greater than 0.5 sec., the maximum inelastic spectral acceleration is obtained by dividing the elastic spectral acceleration by μ; the total inelastic spectral displacement in the same period range is equal to the elastic spectral displacement. For modal periods less than 0.5 sec., the acceleration divisor is $\sqrt{2\mu - 1}$, and the inelastic spectral displacement is $\mu/\sqrt{2\mu - 1}$ times the elastic spectral displacement. This relatively simple modification procedure is based on an extensive study of single degree of freedom elasto-plastic systems (18). Commentary K of NBCC 1977 gives maximum ductility factors for various types of structures, ranging in value from 1 to 4. The modal analysis procedure given in the ATC3 Provisions uses the same static R factors for reducing force response as described previously, thereby ensuring that both the static and modal analysis procedures incorporate ductility in the same manner.

It can be seen that the general effect of having an increased ductility capacity is to reduce the level of the design lateral seismic force V.

Effect of Ductility on Dynamic Response

The previous section has shown that the design procedure incorporates a very simplified approach to incorporate the effect of ductility. The actual effects on dynamic response are much more complex and it is important to recognize that the design procedures may not be very good approximation to reality. A recent study (19) has presented some useful information which indicates the variability of results. In this study, 20 artificial earthquake records of different duration were generated to be compatible with a response spectrum similar to that given in Reference (14). These records were used as input to elastic-plastic single degree of freedom systems having prescribed ductility capacities. Duration had very little effect on the maximum response levels. However, the peak response levels vary considerably with respect to the natural period of the system. These effects are shown for both acceleration and displacement on Figs. 8 and 9, which have been adapted from Reference (19). These figures also show the response spectrum modifications described in NBCC 1977, as discussed in the previous section. From Fig. 8, it can be seen that the elastic/inelastic response ratios (which are the same as inelastic/elastic reduction factors) are much smaller than the code values, indicating that the code will underestimate the actual forces which will be imposed on the structure. It should be noted however, that the value of μ defined in NBCC 1977 is not based on any calculation of actual ductility capacity but is a general value associated with a particular type of structure. This does mean that the true ductility capacity, if calculated, should be something like 50 percent larger than the nominal code value in order for the ductility demand in the actual dynamic response to be achieved by the structure as designed.

Ductility Capacity of Structural Systems

As has been noted previously, the ductility capacities assigned to a particular type of structure in Commentary K of NBCC 1977 are maxima for classes of structure and do not represent the real capacity for any particular structure. There can be considerable variations in these values because the actual ductility capacity of any structure depends on the design of each member and joint, the building configuration and on the workmanship. Unfortunately, there are no simple standard design procedures available which would permit the maximum ductility factor for a particular building to be calculated, although it is possible to do so for simple cantilever shear walls (20). It should be noted that the ductility factor referred to here is the displacement ductility factor (e.g. maximum lateral displacement at the top of a building relative to displacement at first yielding) and not the maximum curvature ductility factor in any member. For most practical structures a much larger curvature ductility capacity is required to achieve a specified level of displacement ductility capacity. For example, in a ten storey ductile moment-resisting frame, assuming a beam sidesway mechanism at collapse, the required maximum curvature ductility factor in the columns is approximately 12 to give a displacement ductility factor of 4 (20).

DEVELOPMENT OF NATIONAL BUILDING CODE OF CANADA

It is appropriate to conclude this paper by giving a brief description of the development of the NBCC seismic loading provisions, beginning with some historical perspectives and concluding with a short discussion of current developments.

The first edition of NBCC was issued in 1941, with the seismic provisions appearing in an appendix; these were based on the concepts presented in the 1937 U.S. Uniform Building Code (UBC). The lateral earthquake force was assumed to act at the centre of gravity of the structure and to have a magnitude given by the product of the building weight and a seismic base shear coefficient. For a building located in the region of highest seismic risk, the value of the base shear coefficient varied from 0.02 to 0.05 depending on the bearing capacity of the soil.

For the second edition of NBCC (1953), the seismic load provisions were updated and placed in the main text. These requirements included two significant new aspects: a Canadian seismic zoning map and consideration of building flexibility in the computation of seismic lateral forces. The 1953 zoning map divided Canada into four "earthquake intensity zones" and remained in force until 1970. The base shear coefficient included the number of storeys N in the denominator (e.g. $C = 0.60/(N + 4.5)$ for buildings in zone 3) as an implicit recognition of the effect of period in modifying the response of the structure during seismic loading. The next edition of NBCC (1960) contained essentially the same seismic loading provisions as the 1953 edition.

The seismic loading provisions of NBCC 1965 were heavily influenced by the 1959 SEAOC Code, which represented the state of the art in earthquake engineering at that time in the U.S. The total seismic lateral force V was given by a series of five multipliers of the building weight W. Three of these (construction type factor, structural flexibility factor and seismic regionalization factor) parallel similar multipliers in the SEAOC Code (although not necessarily having the same numerical values). The other two (importance factor and foundation factor) were not to be found in either the SEAOC Code or the then existing 1941 UBC. The distribution of the lateral force V over the height of the building was such that the force at any storey was proportional to the height of that storey above ground and the weight of the floor, which was as recommended in the 1959 SEAOC Code. In addition to the lateral force definition, the 1965 NBCC specified computation of torsional moments in the horizontal plane. These were based on the then existing Mexican code; the U.S. Codes at that time did not include such explicit torsion requirements. The 1965 NBCC also permitted design loading to be determined by dynamic analysis as an alternative to the simple equivalent static load analysis, provided that "such an analysis is carried out by a person competent in this field of work".

The 1970 edition of NBCC made reference to a completely revised Canadian seismic zoning map, which is still used in NBCC 1977. This zoning map was devised by Milne and Davenport (21) by calculating and contouring the peak horizontal ground acceleration amplitudes that had a mean probability of excedance of 0.01 per annum (A_{100}). The general format of the expression for the lateral force V remained the same as in the previous edition, although the detailed expressions and values for the multipliers were changed. For example, the structural flexibility factor was made a direct function of the fundamental period of the structure, with approximate empirical expressions used to evaluate the period.

The 1975 edition of NBCC incorporated the "assigned horizontal design ground acceleration A", replacing the seismic regionalization factor. Even though the values of A correspond to the zonal values of A_{100} on the zoning map, it should be noted that the specified lateral seismic factor has no direct relationship to the peak forces which would be produced in the structure by an earthquake having a peak horizontal ground acceleration A. In fact the new seismic response factor S, which replaced the previous structural flexibility factor, was calibrated so that the lateral force V would have a value of 20 percent less than the value computed for the same structure in NBCC 1970. In addition, NBCC 1975 permitted dynamic analysis as an alternative to the static load procedure, and gave a detailed description of a recommended dynamic procedure in Commentary K. This dynamic procedure is essentially the same as in NBCC 1977, which has already been discussed in some detail.

The current edition, namely NBCC 1977, has essentially the same seismic loading provisions as the 1975 edition. One major change, which has already been mentioned, is that the dynamically determined seismic base shear should not be less than 90 percent of the base shear determined by the equivalent static load procedure.

A more detailed discussion of the history of the NBCC seismic loading provisions is given in Reference (1), which includes numerical comparisons of the values of the base shear coefficients in the different editions of the code.

Since 1965, the development of the seismic loading provisions for the NBCC has been the responsibility of the Canadian National Committee on Earthquake Engineering (CANCEE). Recommendations for such provisions are made by CANCEE to the Standing Committee on Structural Design, which in turn recommends code provisions to the Associate Committee on the National Building Code, National Research Council of Canada (NRCC).

CANCEE comprises approximately 20 members, drawn from industry, government and universities. They are drawn from the various disciplines which contribute to the overall field of earthquake engineering. All members give their time on a volunteer basis, both to attend meetings and to do the necessary preparatory work, which often involves a considerable amount of applied research. In addition, staff of the Division of Building Research, NRCC, also provide technical and secretarial support for the work of CANCEE.

The next edition of NBCC will be published in 1980 and the proposed changes have already been issued for public comment. The seismic provisions contain no major new developments, but there are a number of specific changes. The most significant of these are:

a. The seismic response factor S is being modified to contain $\sqrt{T}$ in the denominator rather than $T^{1/3}$; this change is being made to provide closer agreement with other methods of predicting the seismic response of a structure.

b. The table defining seismic response factors S_p for parts or portions of buildings is being revised, both in definition of categories and in the value of S_p. In particular, the value of S_p for machinery, fixtures and equipment, pipes and tanks connected to or forming part of a building is being increased from 2 to 10. This particular change is being made to account for amplification of flexible equipment relative to the building, based on actual dynamic response studies.

c. The definition of eccentricity is being changed to clarify the intent of the code and to avoid misinterpretations which lead to incorrect calculation of torsional moments.

In addition to actual code changes, the commentaries dealing with seismic loading and dynamic analysis are being modified slightly in order to improve the understanding of this material.

At the present time CANCEE is continuing with studies and investigations which will be used in making recommendations for future changes in the seismic loading provisions of the NBCC. The aspects being considered include the following"

a) modification of the zoning map to include an additional zone in the higher risk regions of the country,

b) inclusion of other ground motion parameters in addition to peak acceleration, e.g. peak velocity,

c) providing a better agreement between the statically and dynamically determined seismic lateral forces.

d) reexamination of torsional load provisions in the light of the results of recent dynamic studies,

e) reconsideration of the procedures for buildings having setbacks,

f) evaluation of ATC3 Provisions relative to applicability to seismic design in Canada, and

g) development of appropriate requirements for masonry structures in seismic regions.

It should be noted that, at the present time, the above list represents areas of consideration and does not imply that specific changes will be recommended in any or all of these areas.

In addition to the above, CANCEE has for several years been developing a "benchmark structures" programme. In this programme, the properties of a series of buildings actually designed and built in Canada are determined and used to evaluate the effects of proposed code changes. In addition, the structural properties are made available to researchers so that research studies can be done on actual buildings rather than on hypothetical structures. At the present time, CANCEE is developing liaison with the Applied Technology Council in the U.S., which is embarking on a similar project to evaluate the effect of the new ATC3 Provisions on actual building designs.

CONCLUSIONS

This paper has attempted to discuss a number of aspects of earthquake loading and design as applicable to the Canadian situation. It should not be seen as an attempt to completely describe the seismic loading provisions of the National Building Code of Canada, but should be seen primarily as an exposure of a number of significant areas, most of which require ongoing development.

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Table 1 DBE/NBCC Earthquake Response Ratios For Point LePreau Nuclear Power Station

Ground Accelerations: DBE = 0.15g (equivalent)
NBCC = 0.03g

	DBE/NBCC	DBE/(5XNBCC)
Ground Acceleration Ratio	5	1
Containment Building response:		
Base Shear V	12.6	2.51
Overturning Moment M	11.8	2.35
Reactor Building Equipment Response	68.6	13.7

Table 2 Base Shears and Deformation for Different Input Levels, for Elastic Perfectly Plastic Systems

Acceleration Levels	Deformation	Base Shear
$a = A_e$	$\delta_e = \frac{A_e}{A_y}\delta_y$	$V_e = \frac{A_e}{A_y} V_y$
$A_e < a < A_y$	$\delta = \frac{a}{A_y}\delta_y$	$V = \frac{a}{A_y} V_y$
$a = A_y$	$\delta = \delta_y$	$V = V_y$
$A_y < a < A_u$	$\delta = \frac{a}{A_y}\delta_y$	$V = V_y$
$a = A_u$	$\delta = \delta_u = \frac{A_u}{A_y}\delta_y$	$V_u = V_y$

Table 3 Abbreviated Definitions of K Coefficients in NBCC 1977

(Full definitions are given in Table 4.1.9.A of NBCC 1977)

Type of Structure	Value of K
1. Buildings with a ductile moment-resisting space frame with the capacity to resist the total required force.	0.7
2. Buildings with a dual structural system consisting of a complete ductile moment-resisting space frame and ductile flexural walls.	0.7
3. Buildings with a dual structural system consisting of a complete ductile moment-resisting space frame and shear walls or steel bracing.	0.8
4. Buildings with ductile flexural walls or with other forms of ductile framing systems.	1.0
5. Buildings with a dual structural system consisting of a complete ductile moment resisting space frame with masonry infilling.	1.3
6. Buildings, other than above, of continuously reinforced concrete, structural steel, and reinforced masonry shear walls.	1.3
7. Buildings of unreinforced masonry.	2.0

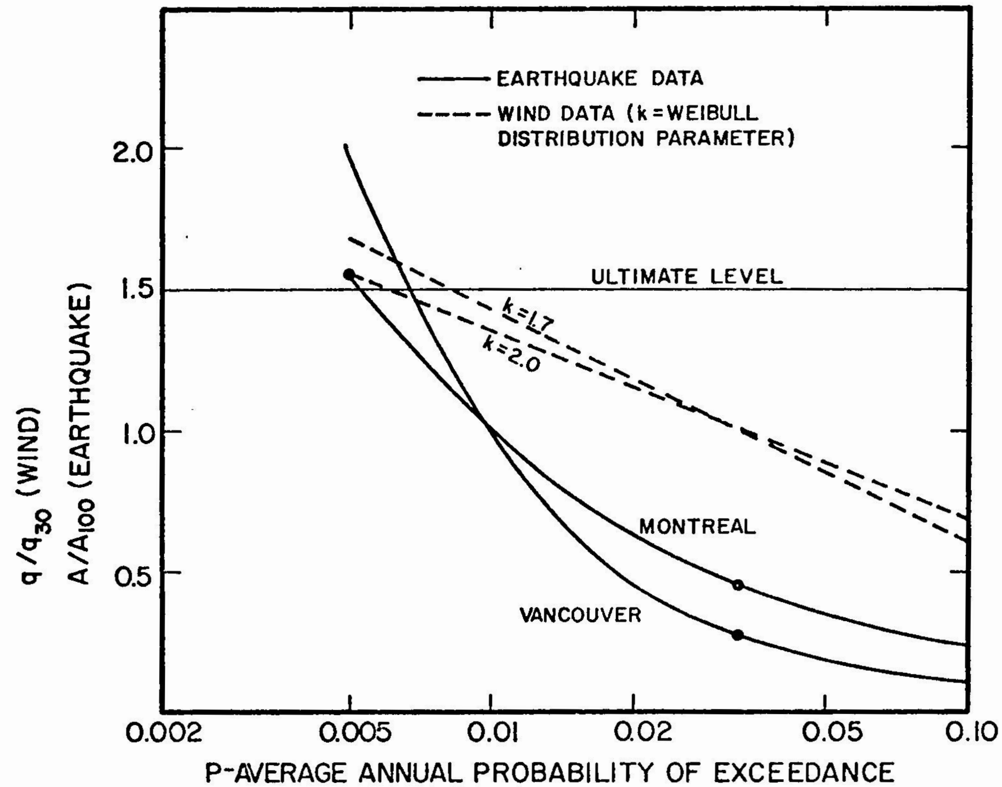

FIGURE 1. COMPARISON OF EARTHQUAKE AND WIND LOADING PARAMETER PROBABILITIES

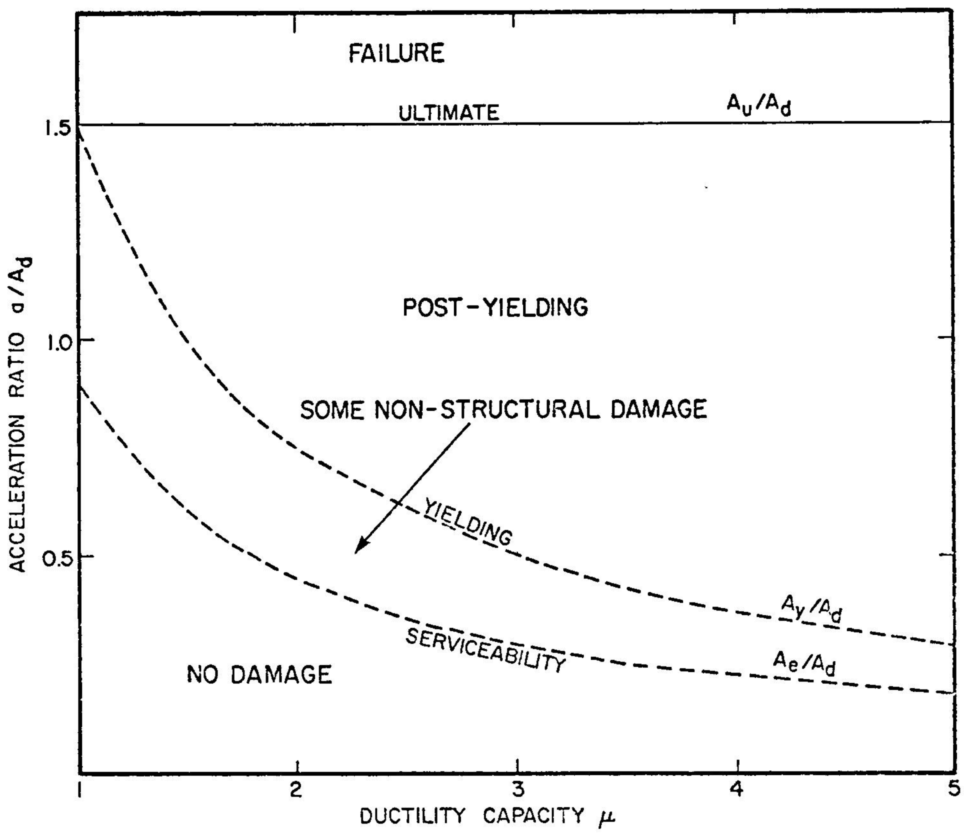

FIGURE 2. STRUCTURAL BEHAVIOUR RELATIONSHIPS (β=0.60 AND α_Q=1.5)

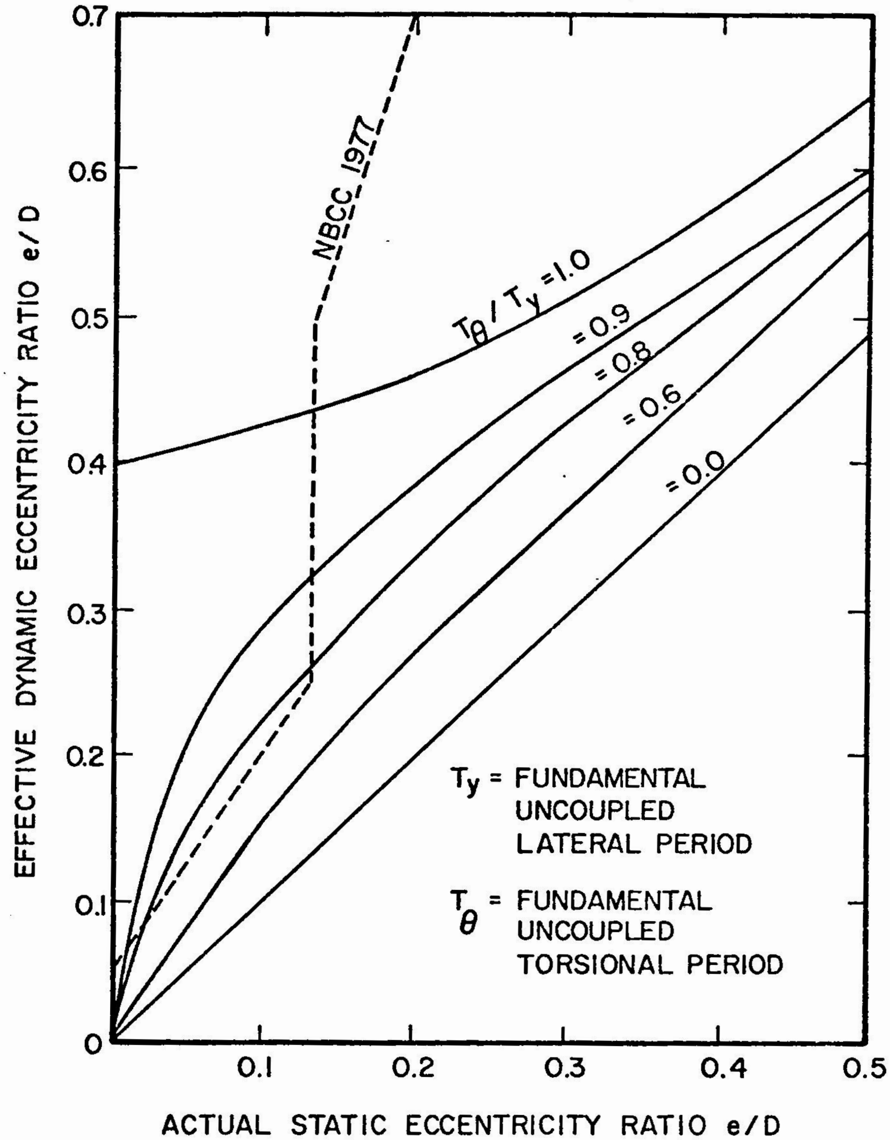

FIGURE 3
MODAL COUPLING EFFECT ON ECCENTRICITY
FOR UNIFORM FLEXURAL CANTILEVER STRUCTURE

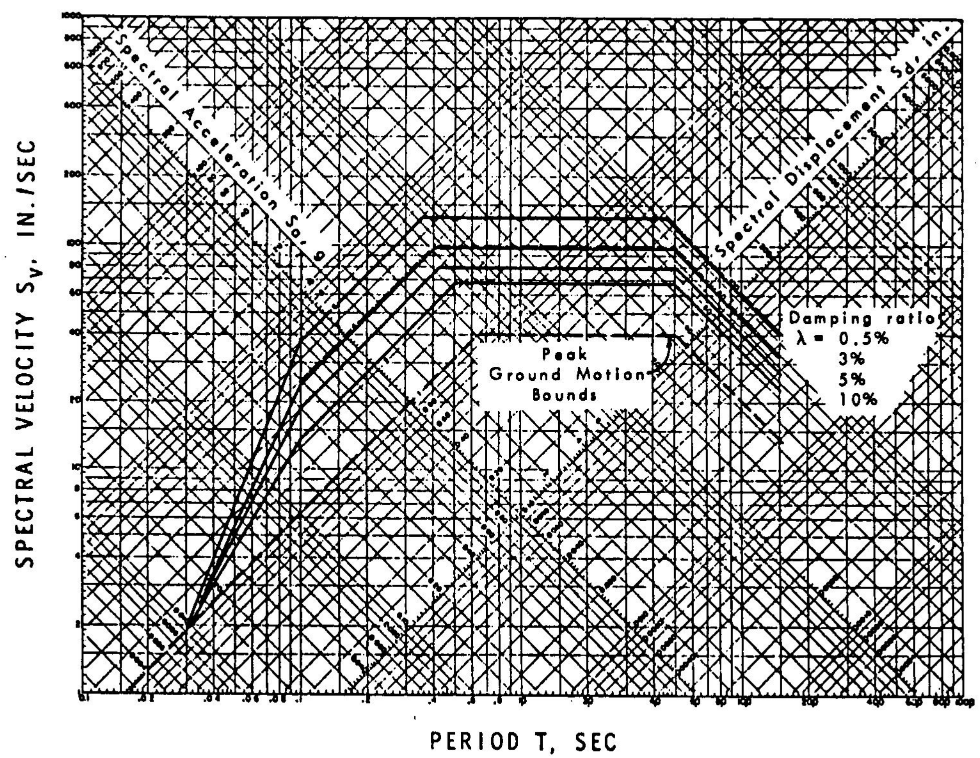

FIGURE 4. ELASTIC AVERAGE RESPONSE SPECTRA FOR 1.0g MAX. GROUND ACCELERATION (FROM COMMENTARY K, NBCC 1977)

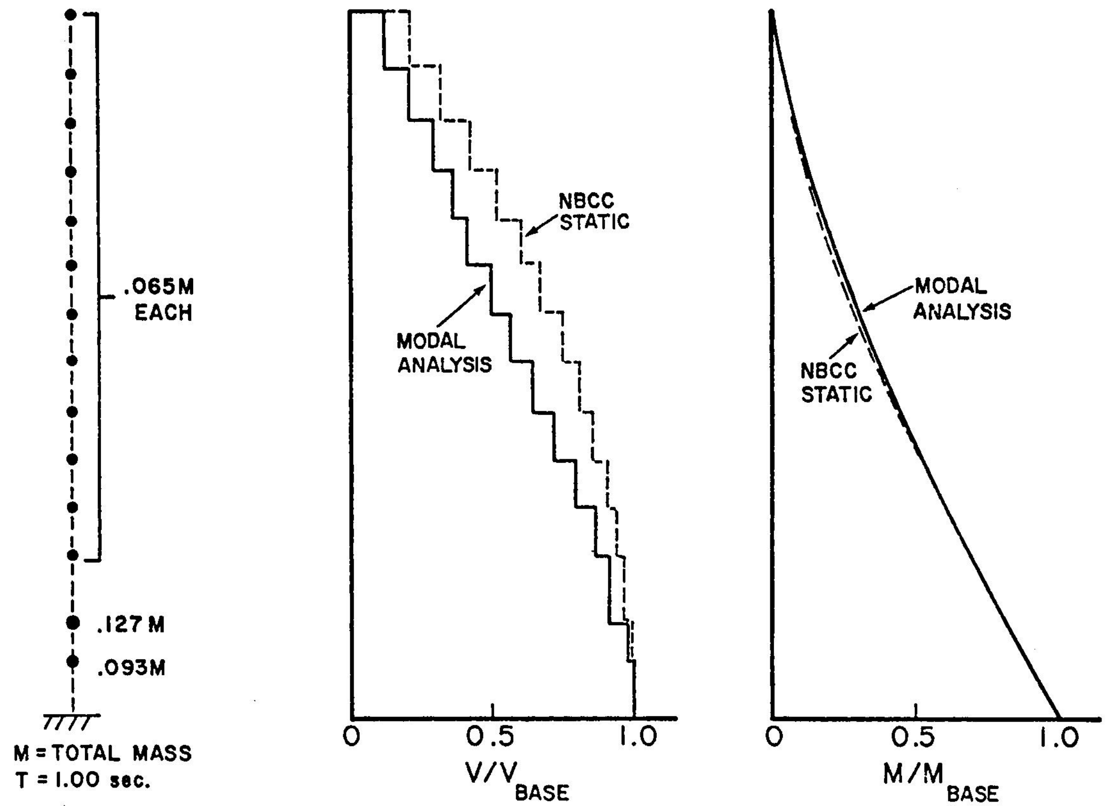

FIGURE 5. SEISMIC ANALYSIS OF FOURTEEN STOREY R C FRAME STRUCTURE

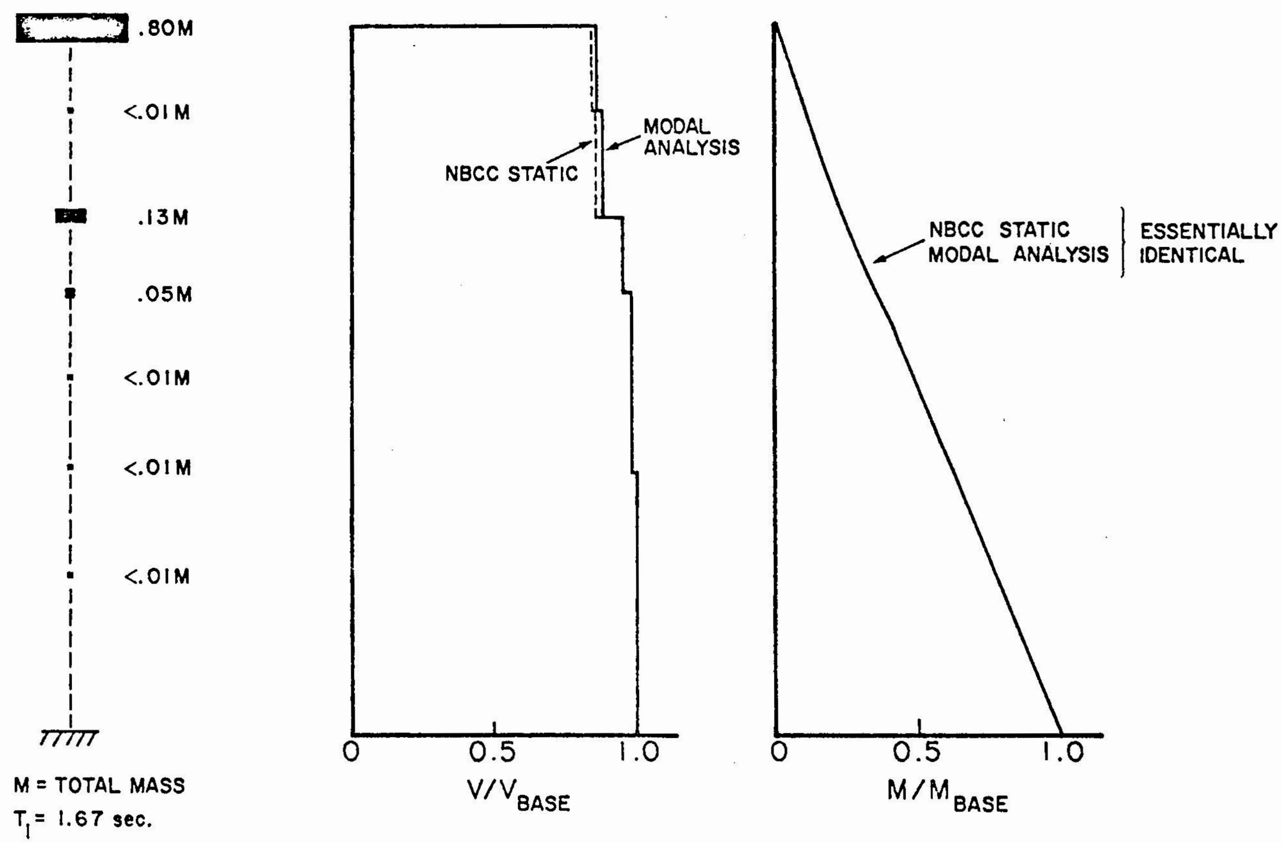

FIGURE 6. SEISMIC ANALYSIS OF INDUSTRIAL STEEL FRAME STRUCTURE

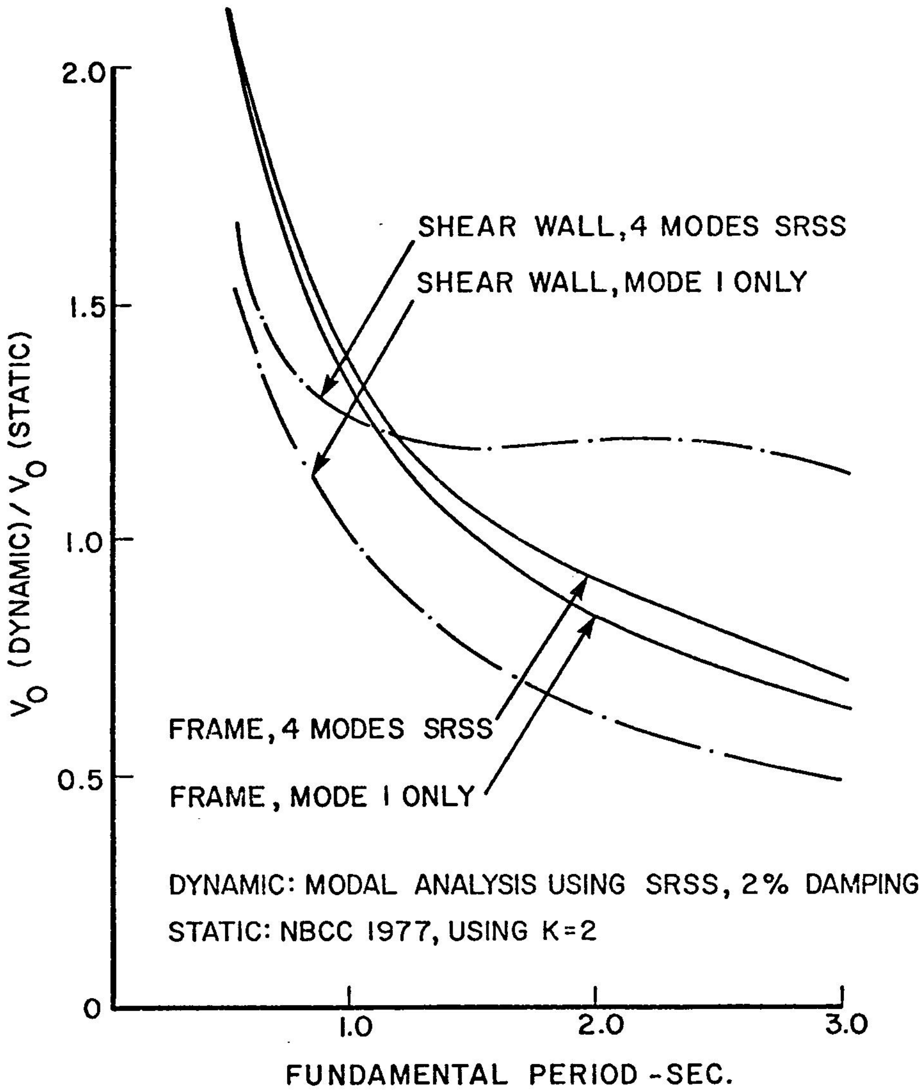

FIGURE 7. VARIATION OF ELASTIC BASE SHEAR

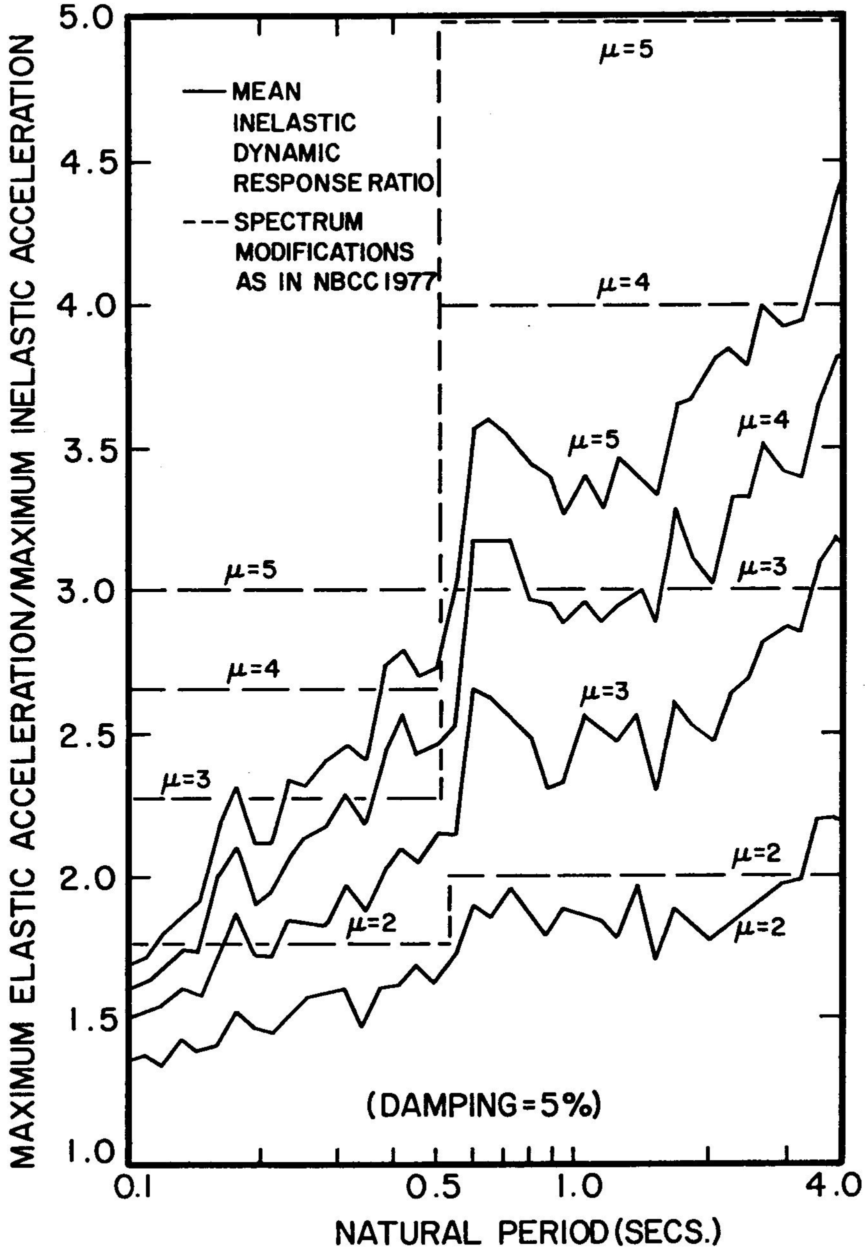

FIGURE 8. INELASTIC ACCELERATION RESPONSE RATIOS (ADAPTED FROM REFERENCE 19)

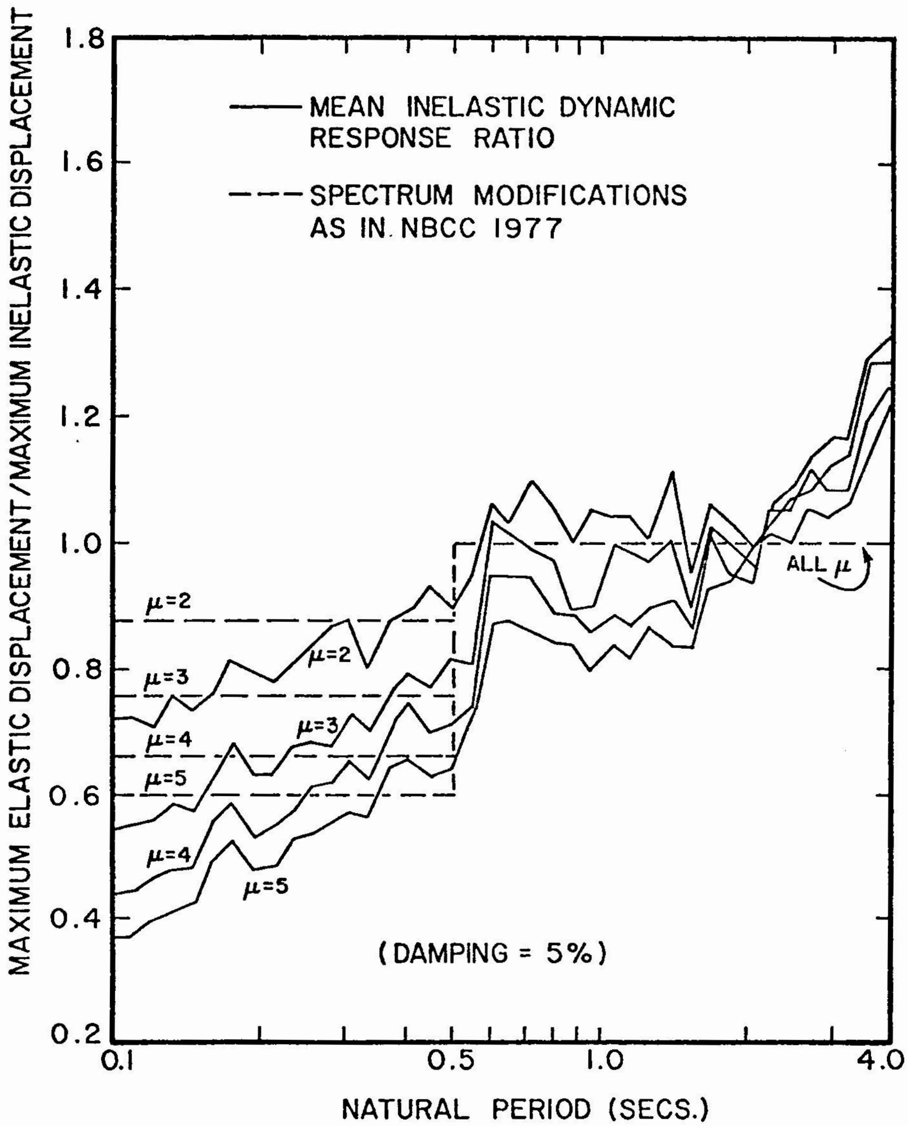

FIGURE 9. INELASTIC DISPLACEMENT RESPONSE RATIOS (ADAPTED FROM REFERENCE 19)