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Earthquake Loading in the Ultimate Limit States Design of Bridges and Structures

L.G. Jaeger[1], A.A. Mufti[2] and J.P. Newhook[3]

ABSTRACT

In Canada, an earthquake intensity which has a 10% probability of exceedance at a given site during a 50-year design live of a structure is becoming widely regarded as a norm. This "NBC" earthquake has a nominal return period N = 475 years. However, in certain cases the design life of the structure is stipulated to be 100 years; the retention of a 10% probability of exceedance during this longer life then leads to an earthquake having a nominal return period of N = 1000 years.

This paper discusses earthquake loading in the design of highway bridges in light of recent seismic design criteria established for the Northumberland Strait Crossing Project. The "NBC" earthquake for bridges is associated with a safety index of $\beta = 3.5$. This paper presents a method for establishing the earthquake loading for a design life other than fifty years and for a safety index other than $\beta = 3.5$. New relationships linking the probability of exceedance of the factored earthquake to the probability of exceedance of the unfactored earthquake and linking the probability of exceedance of the factored earthquake to the notional probability for the safety index, β, are discussed. Using the strong ground motion data for the Northumberland Strait Crossing site, examples of multipliers of the "standard NBC" earthquake for various design lives and safety indices are derived. The implications of the calculated earthquake loading on establishing a response spectrum are also discussed. If the proposed method is adopted, it is demonstrated that the provisions of the Ontario Highway Bridge Design Code for earthquake loading and the provisions of the National Building Code for Canada for earthquake loading are mutually consistent.

INTRODUCTION

In the design of highway bridges using limit states design methods, important concepts are those of the design life of the bridge and the safety index to which it is being designed. This paper begins with a brief review of the limit states design process as such, and then goes on to suggest a way in which earthquake loading may be treated within that design process.

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[1]Vice-President, Vaughan International Consultants Ltd., Halifax, N.S. and Research Professor of Civil Engineering and Applied Mathematics, Technical University of Nova Scotia, P.O. Box 1000, Halifax, N.S. B3J 2X4.

[2]Director, The Nova Scotia CAD/CAM Centre and Professor of Civil Engineering, Technical University of Nova Scotia, P.O. Box 1000, Halifax, N.S. B3J 2X4.

[3]Design Structural Engineer, Vaughan Engineering Associates Ltd., Halifax, N.S. B3J 2Z1.

FACTORED LOADS AND FACTORED RESISTANCES

The bridge designer does not normally deal with the statistical distributions of load and resistance; but rather, the designer uses an inequality of the form.

$$\sum_m \alpha_m L_m \leq \sum_n \phi_n R_n \qquad [1]$$

In words, the inequality [1] says "the total of the factored loads must not exceed the total factored resistance". The load factors α_m are greater than unity and the performance factors ϕ_n are less than unity. Taken together with the values of <u>unfactored</u> loads and resistances, this inequality reflects the selected safety index β.

It is noted that the unfactored nominal load terms are not themselves mean values for the type of load concerned; for example, the live load truck used in the Ontario Highway Bridge Design Code is not the "average truck" that goes along the highway but something close to an upper bound. Conversely, the nominal resistance of a structural component is not the expected value but, something smaller - in the case of wood components, for example, the fifth-percentile value may be used as the nominal.

EARTHQUAKE LOAD AS A COMPONENT OF THE TOTAL FACTORED LOAD

<u>Earthquake provisions of the Ontario Highway Bridge Design Code</u>

The inequality [1] given above must be satisfied for various combinations of load. Thus, for example, the Ontario Highway Bridge Design Code (OHBDC) (Third Edition, 1991) gives a table of load factors (Table 2-5.1(a), OHBDC, 1991) for various load combinations which must be considered.

In the load combination which involves earthquake loading, the earthquake load is given as $1.3Q$, where Q is the earthquake which has a 10% probability of exceedance during the 50-year design life of a bridge. It is noted that

(i) the OHBDC takes a design life of 50 years;
(ii) the earthquake Q defined above is becoming increasingly accepted as a norm in Canada; it is the "normal National Building Code Earthquake" (NRC, 1990).

For bridge design, the combination of safety index $\beta = 3.5$ and design life of the bridge $Y = 50$ years is the most usual one, and the one for which the OHBDC has been calibrated.

<u>Earthquake Provisions for Other Safety Indices and Other Design Lives</u>

It sometimes happens that a bridge structure must be designed for a safety index that is other than 3.5, and/or for a design life that is other than 50 years. An example is the Northumberland Strait Crossing, often referred to as the "PEI Fixed Link", for which a safety index $\beta = 4.00$ is stipulated for multi-load-path members and 4.25 for single-load-path members; the design life of the structure is $Y = 100$ years (PWC, 1988). One is therefore led to consider what earthquake loading for $\beta = 4.00$ or 4.25 and $Y = 100$ years is consistent, from a probability standpoint, with the existing OHBDC provisions.

Keeping the Safety Index Constant and Varying Design Life

Table 1 gives earthquake intensity related to annual probability of exceedance for the PEI site; in this table, the information for the first four columns is derived from that supplied by Dr. John Adams (1994) of EMR Canada, whilst the last two columns have been obtained by extrapolation.

Table 1: Earthquake intensity related to annual probability of exceedance for the PEI site

Probability of Annual Exceedance (p)	0.0100	0.0050	0.0021	0.0010	0.007	0.004
Probability of Exceedance in Ten Years (p_{10})	9.6%	4.9%	2.1%	1.0%	0.7%	0.4%
Probability of Exceedance in Fifty Years (p_{50})	39.5%	22.2%	10.0%	4.9%	3.4%	2.0%
Probability of Exceedance in One Hundred Years (p_{100})	63.4%	39.4%	19.0%	9.5%	6.8%	3.9%
Acceleration Level (Fraction of g)	0.031	0.043	0.064	0.086	0.100	0.120

It may be noted that in Table 1, only the last row is site-specific. The other rows are related by the formula

$$py = 1 - (1-p)^{y} \qquad [2]$$

where p = annual probability of exceedance
py = probability of exceedance in Y years

We now examine the earthquake loading "1.3Q" and investigate whether this expression is valid for other values of design life Y years, provided that "Q" is regarded in every case as the earthquake that has a 10% probability of exceedance during the design life concerned. This way of thinking amounts to posing the question "If the OHBDC had been based upon a different design life, say 100 years, would the earthquake loading term 1.3Q still be valid, provided that Q is now regarded as the earthquake that has a 10% probability of exceedance during this different design life, it being borne in mind that the safety index β is being kept at the value of 3.5?"

To answer this question, one must define a relationship between the first and last rows of Table 1, along with suitable notation. If P = annual probability of non-exceedance = (1-p), then equation [2] above may be written

$$P_y = P^y \qquad [2a]$$

where P_y is the probability of non-exceedance in Y years.

The relationship between probability and intensity of an earthquake adopted by Milne and Davenport (1969) with a change of notation may be written

$$-\log_e P = \left(\frac{G}{C}\right)^s \qquad [3]$$

where G is acceleration intensity and C and s are site-specific constants. On now introducing subscripts U and F for unfactored and factored earthquakes respectively, and α for the load factor, equation [2a] and [3] give

$$\begin{aligned} -\log_e P_{YU} &= Y \log_e P_U = Y\left(\frac{Q}{C}\right)^s \\ -\log_e P_{YF} &= Y \log_e P_F = Y\left(\frac{\alpha Q}{C}\right)^s \end{aligned} \qquad [4]$$

from which

$$\frac{\log_e P_{YF}}{\log_e P_{YU}} = \alpha^s \qquad [5]$$

It is becoming increasingly common to work with probability of exceedance, p, rather than of non-exceedance, P. In terms of probabilities of exceedance, equation [5] may be written

$$p_{YF} = 1 - (1 - p_{YU})^{\alpha^s} \qquad [6]$$

Equation [5] is significant. It means that the ratio

$$\frac{\text{Probability of Non-Exceedence of the Factored Earthquake over Y years}}{\text{Probability of Non-Exceedance of the Unfactored Earthquake over Y years}}$$

is independent of the period Y years. Hence, if Q is defined to be the unfactored earthquake for which P_{YU} is 0.9000 (i.e. a 10% probability of exceedance) then P_{YF} is readily found from equation [5] and is invariant with the period Y years. If the load factor is 1.3 then for the PEI site P_{YF} is readily found to be 0.9465, (i.e. a 5.35 % probability of exceedance), no matter what the value of Y.

For the PEI site we then have, no matter what the design life, a probability of exceedance of the already factored earthquake = 0.0535 and a notional probability = 0.00023 for β = 3.5 (see Table 2).

Table 2: Notional Probability for Various Safety Index Values

β	$\phi(-\beta)$
0	0.5
1	1.6×10^{-1}
2	2.3×10^{-2}
3	1.35×10^{-3}
4	3.2×10^{-5}

We now link these two probabilities by an indicial relationship

$$0.0535 = (0.00023)^{\gamma}$$

from which $\gamma = 0.350$.

Keeping the Design Life Constant and Varying the Safety Index

If the safety index is now changed from 3.5 to 4.0 the notional probability of exceedance changes from 0.00023 to 0.000032. Postulating the same indicial relationship $\gamma = 0.35$ for earthquake probability in relation to this change of notional probability one obtains a factor of $(\frac{0.000032}{0.00023})^{0.35}$ which is 0.501, (say 0.50 to design accuracy) as the factor by which earthquake probability will change; one is thus led to postulate that for $\beta = 4.0$ the unfactored earthquake Q should be taken as one having a probability of exceedance of 5% during the design life concerned, rather than the 10% exceedance associated with $\beta = 3.5$.

Hence a loading αQ is postulated to be valid for various β and Y, provided that Q is taken to be the earthquake event which has a probability of exceedance in Y years of (10λ)% where λ is the quantity

$$\lambda = \left[\frac{\text{Notional Probability of Exceedance for the given } \beta}{\text{Notional Probability of Exceedance for } \beta = 3.5}\right]^{0.35}$$

$$= \left[\frac{q(\beta)}{q(3.5)}\right]^{0.35}$$

Values of λ are given in Table 3.

Table 3: Values of λ for various β

β	3.0	3.25	3.50	3.75	4.00	4.25
λ	1.88	1.37	1.00	0.714	0.501	0.345

It is noted that λ is not site-specific but depends solely on the properties of the normal distribution curve. It is further noted that the load factor α is independent of both β and Y.

Using Tables 1 and 3, it is a straightforward matter for the PEI site to find the earthquake Q and thence 1.3Q for any pairing of β and Y. For example, for $\beta = 3.00$ and Y = 100 years, one finds $\lambda = 1.88$ from Table 3 and then seeks a probability of 10λ(= 18.8%) in the fourth row of Table 1. By interpolation this value occurs between the third and fourth columns, at an acceleration which is 0.065g. Multiplying by 1.3 then gives the factored earthquake intensity as 0.85g. Proceeding in this way one obtains the values shown in Table 4.

Table 4: Factored Earthquake Intensity for Various β and Y, Given as Decimal Fractions of g

Design Life β	Y years 10	50	100
3.00	0.030	0.060	0.085
3.50	0.040	0.083	0.111
4.00	0.056	0.110	0.140

It is sometimes helpful to the designer to give the same information in the form of the return period $(=\frac{1}{p})$ of the earthquake which can be used immediately as the earthquake loading term in the factored load side of inequality [1], for the relevant pairing of β and Y. For the PEI site the result is that shown in Table 5.

Table 5: Return Period $(\frac{1}{p})$Years for Factored Earthquake Loading for Various β and Y

β	Y years 10	50	100
3.0	95	425	950
3.5	180	900	1870
4.0	370	1870	3700

If preferred, the information contained in Table 4 may be given in the form of factors by which the "standard NBC" earthquake must be multiplied. The result is shown in Table 6.

Table 6: Multipliers of the Standard NBC Earthquake

β	Y years 10	50	100
3.0	0.47	0.95	1.31
3.5	0.64	1.30	1.72
4.0	0.89	1.72	2.19

Many designers might prefer to use Table 6 for three reasons:

(i) the standard NBC earthquake is rapidly becoming a norm of design, and the use of multipliers of this norm may be give the designer a valuable "feel" for the design.

(ii) the use of the multipliers does away with the need to extrapolate earthquake behaviour at a given site to extremely rare events.

(iii) it enables a quick estimate to be made of the "additional multiplier" for any (β,Y) combination as compared to normal design. For example, with β = 4.00 and Y = 100 years we have a multiplier of 2.19, compared with a multiplier of 1.3 for normal design (see middle entry on the second row). This means that, compared to normal design, an additional multiplier of $\frac{2.19}{1.30} = 1.68$ is applied to the loading.

INCLUSION OF THE AMPLIFICATION FACTORS

The reasoning given above is directed towards establishing a ground motion which carries with it an acceptably small probability of exceedance. It is useful and relevant to consider also the amplification factors by which the ground motion is multiplied in order to give a response spectrum for a given level of damping. Such amplification factors are usually given as either "mean" values (i.e. with 50% expectation of being exceeded) or "mean plus one standard deviation" (i.e. with about 15% probability of exceedance). The adoption of "mean plus one standard deviation" as compared with "mean" thus introduces a further probability multiplier of about 0.32.

If one takes the view that it is the probability of exceedance associated with the response spectrum that is ultimately decisive, then, from a probability standpoint, the following three approaches need to be considered.

(a) Give ground motion which itself has the necessary small probability of exceedance, and amplification factors at their "expected" or "mean" values, so that the probability of exceedance of the response spectrum is one half that of the ground motion.

(b) Give ground motion which itself has the necessary small probability of exceedance and set the amplification factors at "mean plus one standard deviation", in which case there is an additional "bonus of improbability" arising from the amplification factors.

(c) Give ground motion which, in combination with amplification factors of "mean plus one standard deviation" will give a response spectrum having the same probability of exceedance, as in (a) above, or better.

Thus, for example, if one takes the choice (c) in relation to the PEI site, one could take a ground motion acceleration of 0.12g (for which p_{100} = 0.0421) together with an acceleration amplification factor of 3.0 (see below) and have the same safety as one would have on taking ground motion acceleration for which p_{100} = 0.0134, along with the mean values of the amplification factors.

If one adopts choice (c), then it is wise to have amplification factors which represent site-specific behaviour as closely as possible. Newmark and Hall (1982) for California earthquakes give factors of 2.70 for acceleration and 2.30 for velocity. However, Naumoski, Heidebrecht and Wang (1991) have noted that higher factors should be used for Eastern Canada. They note a peak factor of 3.30 for acceleration, for example, and scrutinizing of their graphs across the range of applicable frequencies suggests an overall amplification factor for acceleration of about 3.0 or a little higher.

POSSIBLE WIDER VALIDITY OF THE PATTERNS OF BEHAVIOUR

Tables 5 and 6 have been drawn up for the PEI site. It is relevant to bear in mind that they may change not too markedly if one considers other sites. Thus, two sites may have "standard NBC earthquakes" which differ from one another in acceleration intensity by a factor of 5 or more; however, it is possible that the ratio of acceleration level for say 1000 year return to that for 475 year return may be much more nearly the same for one site as the other. Tables 5 and 6 are based upon such ratios; it would be a worthwhile activity to take site-specific data from a number of other sites, to construct these tables for each site in turn, and examine their variability.

CONSISTENCY BETWEEN NBC AND OHBDC

The National Building Code of Canada (NBC) is based upon an earthquake loading Q for a safety index of $\beta = 3.00$, whilst the Ontario Highway Bridge Design Code (OHBDC) gives an earthquake loading 1.3Q for a safety index $\beta = 3.50$. Both codes are based on Y = 50 years. Table 6 above shows that, for the PEI Site, and using the suggested indicial relationship, a loading of 0.95Q for $\beta = 3.00$ and Y = 50 years is consistent with a loading of 1.3Q for $\beta = 3.50$ and Y = 50 years. The difference between 0.95Q and Q is barely detectable to design accuracy; thus Table 6 suggests a good level of consistency between the NBC and OHBDC. This is encouraging and also lends support to the indicial relationship itself.

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