THE MCE (MAXIMUM CREDIBLE EARTHQUAKE) —
AN APPROACH TO REDUCTION OF SEISMIC RISK

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It is the responsibility of the Regulatory Body (in Canada, the AECCB) to ensure that radiological risks resulting from the effects of earthquakes on nuclear facilities, do not exceed acceptable levels. In simplified numerical terms this means that the frequency of an unacceptable radiation dose must be kept below $10^{-6}$ per annum. Unfortunately, seismic events fall into the class of external events which are not well defined at these low frequency levels. Thus, design earthquakes have been chosen, at the $10^{-3} - 10^{-4}$ frequency level, a level commensurate with the limits of statistical data. There exists, therefore, a need to define an additional level of earthquake. A seismic design explicitly and implicitly recognizes three levels of earthquake loading; one comfortably below yield, one at or about yield, and one at ultimate. The ultimate level earthquake, contrary to the first two, has been implicitly addressed by conscientious designers by choosing systems, materials and details compatible with postulated dynamic forces. It is the purpose of this paper to discuss the regulatory specifications required to quantify this third level, or Maximum Credible Earthquake (MCE).

The intensity and duration of the radiological exposure following an earthquake is a function of the following three elements: 1) the severity of the earthquake; 2) the resistance offered by the engineered structures and equipment to the seismic event; and 3) the release and dispersion of the radiological effluent to members of the public if the resistance is insufficient. With the present incomplete knowledge of tectonic processes in eastern Canada, there is no alternative but to extrapolate, on the basis of the best available scientific evidence, to the low frequency events required. This extrapolation has been carried out using a Monte Carlo simulation technique. The result is an ensemble of earthquakes catalogued as to near and far field events. To these representative time histories have been assigned. Such an exercise indicates the relative importance of near and far events and shows, for example, that for eastern Canada, the random, background earthquake occurring close to the plant would cause the major seismic threat.

The nuclear power plant (NPP) must be built to survive the effects of the ensemble of earthquakes defined in element #1. The use of design in the inelastic range is explored by the use of coupled, two-degrees of freedom models, employing softening (hysteretic) and hardening (gap-impact) restoring functions. A "matrix" approach is employed to investigate, in a systematic manner, non-linear effects of equipment only, structure only, and structure and equipment. It is shown that, except for certain anomalies such as non-linear torsional coupling and internal resonance conditions, the use of non-linear properties (provided that the design has incorporated the required ductility and continuity properties) will ensure the survival of passive structures and systems to the ensemble of earthquakes that specify the MCE. Finally, the third element is discussed and the consequences are placed into perspective with other nuclear plant caused and general societal risks.
1. Introduction

All buildings in Canada must meet the minimum seismic design requirements of the Canadian National Building Code. The early Nuclear Power Plants (NPP) met these requirements. In addition to the requirements of structural "non-collapse" demanded by the code, NPP were also required to remain operationally functional. The "functional" requirement opened a new field of earthquake engineering which generated and is still generating a host of new seismic design requirements\(^1\). In Canada these design requirements are being addressed in the new Canadian Standard Association Code N289.

Realizing that there are only about 300 years of earthquake records available in Eastern Canada\(^2\), the committee in charge of writing the code has felt that it would be unrealistic to extrapolate the limited data lower than a frequency of about \(1 \times 10^{-3}\) events/year. To demonstrate that the frequency of unacceptable consequences is actually much lower than this, it is proposed to take advantage of the safety margins included in every step of the engineering design process. These safety margins have been listed in reference \(3\) where, for a particular level of design basis earthquake (chosen at a frequency of \(1 \times 10^{-3}\) events/year), an argument is made that the overall frequency of radioactivity releases exceeding acceptable limits will be less than \(1 \times 10^{-7}\) events/plant-year.

Quantifying the design safety margins is one way to attempt to determine the probabilities of radioactive release; it is however a most ambitious way and in some sense a way of last resort which does not follow normal engineering practice. A more direct approach is to subject the critical systems of the nuclear power plant to the Maximum Credible Earthquake (MCE) and show by realistic analyses and testing that critical safety functions of the NPP can still be performed; this may require dedicated, "hardened" systems specifically designed for the MCE.

2. The Approach

The intensity and duration of the radiological exposure following an earthquake is a function of the following three elements:

Element 1 - The severity of the earthquake.

Element 2 - The resistance offered by the engineered structures and equipment to the seismic event.

Element 3 - The release and dispersion of the radiological effluent to members of the public.

Element 1 is a function of the seismology and tectonics of the site; element 2 is a function of engineered protection; element 3 is a function of the meteorology and population density. Risk to the public is a function of these 3 elements. The risk is considered acceptable if the calculated doses meet the AECB's reference dose limits\(^4\).

3. Element 1 - The Frequency and Severity of Earthquakes

Seismic severity increases continuously as the frequency of the event decreases. In principle, it is possible to raise the engineered resistance to meet the intensity of loading, although in the limit, massive fault displacement for which there is no engineering solution could be postulated to occur at the site. Such faulting has not been observed in eastern Canada, and therefore we assume it sufficient to extrapolate, based on the best available scientific evidence, to the low frequency events required. This extrapolation can be carried out for a hypothetical site using a Monte Carlo simulation technique\(^5\).

The results of the analysis are best displayed by a graph of cumulative frequency vs site acceleration as shown in Figure 1. Four curves are superimposed to show the sensitivity of the plots to variation in the assumption of maximum magnitude. Figure 2 displays all seismic

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events which cause a site acceleration greater than 0.20 g. Plotted in Figure 2 is the epicentral distance against site acceleration for those events with maximum magnitude M=7. Figure 1 illustrates two difficulties in low frequency projection. The first difficulty is numerical and concerns the number of Monte Carlo trials used. The uncertainty band for each individual curve broadens as the frequency decreases since there exists an economic constraint on the number of trials that can be carried out. The second difficulty is much more profound. The four curves show the spread of results possible simply by varying the assumed maximum magnitude. Such large sensitivity coupled with increasing uncertainty as the frequency decreases presents a rather severe challenge to the ingenuity of the analyst. This is the "Achilles heel" of probabilistic seismic analysis of nuclear power plants.

Specification of the MCE requires realistic descriptions of a representative range of seismic events. A statistically significant sample of earthquake motions for each of the possible source mechanisms identified in the seismological investigation is essential; representative time histories are required for non-linear analysis. For the purpose of this paper we have composed a sub-set of two earthquakes to describe the MCE by choosing the first 8 seconds of El Centro to represent a far-source earthquake and by creating, based on references 6 and 7, an artificial earthquake with a duration of one half second to represent the near-field earthquake. Both earthquakes were given a maximum acceleration of twice the Design Basis Earthquake (DBE); structures and equipment designed to the DBE can be expected to approach but remain within yield.

4. Element 2 - The Engineered Resistance

The nuclear power plant must be built to survive with a high degree of confidence the effects of the ensemble of earthquakes defined as the MCE in element 1 such that shut-down of the reactor is not impaired and decay heat from the reactor and spent fuel bays is removed as required. Proven procedures to analyze into the ductile plastic zone for passive structures and piping systems are vital to obtaining a realistic assessment of dynamic response. While it is generally accepted that non-linearities will cause beneficial results due to increased energy absorption, and mismatch of frequencies, there are instances where non-linear effects can be detrimental. To note one example, non-linear coupling between the lateral translational motions can induce and reinforce torsional motions which may lead to an unexpected mode of failure.

To investigate the use of design in the inelastic range the response of a coupled, two-degree-of-freedom model, employing softening (hysteretic) and hardening (gap-impact) restoring functions as shown in Figure 3 can be employed. The thrust of the numerical experiments reported herein is to explore the response of this structural system when it is subjected to an earthquake excitation twice that of its design value. It will be assumed that at its design value the structural response is linear but very close to yield.

The response of this system has the 4 choices as shown:

<table>
<thead>
<tr>
<th>EQUIPMENT</th>
<th>RESPONSE CHOICES</th>
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<tbody>
<tr>
<td>Linear</td>
<td>1</td>
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<tr>
<td>X</td>
<td>-</td>
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<td>Non-linear</td>
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<tr>
<th>STRUCTURE</th>
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<tr>
<td>Linear</td>
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<tr>
<td>Non-linear</td>
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</table>
The problems of non-linear analysis can be appreciated by looking at this very simple table. The term "linear" is a reasonably restricted term and a consensus could probably be obtained amongst design engineers as to what is meant by a "linear" analysis. However, the negative term "non-linear" applies to all things other than a "linear" analysis - the word is unbounded in its definition and difficult to deal with in an engineering way. For the purpose of this paper, results for response choice 2 will be shown for hysteretic and gap-impact restoring functions and for response choice 3 for the hysteretic function only. These non-linear responses will be compared against response choice 1. Response choice 4 is left for future considerations.

Non-linear (Hysteretic) Effects of Equipment

Referring to Figure 3, the equations of motion of this system, with a non-linear restoring function for the secondary equipment, may be written as:

\[ \dddot{X}_1 + c_1 \dddot{X}_1 + k_1 X_1 - R(\dot{X}) = -m \dddot{U} \]  
\[ \dddot{X}_2 + c_2 \dddot{X}_2 + R(\dot{X}) = -m \dddot{U} \]  

where \( R(\dot{X}) \) represents the non-linear restoring force of the equipment mounting. The co-efficients \( c_1 \) and \( c_2 \) are included as a matter of convenience to add viscous damping. Employing the substitutions,

\[ u_1^2 = \frac{k_1}{M} \]  
\[ u_2^2 = \frac{k_2}{m} \]  
\[ m = \epsilon M \]  
\[ \dddot{u} = g\phi(t) \quad (g = 32.2 \text{ ft/sec}^2) \]  
\[ \tau = \frac{1}{k_1} \]  
\[ \dddot{x}_1 = \frac{x_1}{(i=1,2)} \]  
\[ \dddot{x}_2 = \frac{g}{u_2^2} \left( \frac{\delta - \dddot{x}_2}{\tau^2} \right) \]  
\[ f = \frac{R(\dot{x})}{k_2 u_2^2} \quad \text{non-linear force} \]  
\[ \tau = \frac{u_2}{u_1} \]  

leads to the equations,

\[ \dddot{u}_1 + 2 \epsilon \dddot{u}_1 + \dddot{u}_1 - \epsilon^2 \tau f = -\phi(\tau/\omega_1) \]  
\[ \dddot{u}_2 + 2 \epsilon \dddot{u}_2 + \dddot{u}_2 + \tau^2 f = -\phi(\tau/\omega_1) \]  

Equations 3 form a coupled, non-linear system whose response can in general only be determined by direct numerical integration. To facilitate the study the structural parameters are chosen as follows: (a) the mass ratio \( \epsilon \) is fixed (\( \epsilon = 0.001 \)), (b) the frequency ratio \( \tau \) has 8 discrete values in the range 0.5\( \leq \tau \leq 2 \), (Cases 1 to 8 in Tables 1 and 2) (c) \( u_1 \) is fixed to correspond to 10 Herz and, (d) the hysteretic function \( f \) is either bi-linear (\( U = 0.5 \)) or fully plastic (\( U = 1.0 \)) as shown in Figure 4 and Figure 7 respectively. The parameters of interest are the absolute accelerations and relative displacements of the equipment. Their actual values, however, are of little interest. What is of interest is the influence of the non-linearities on the response as the excitation increases and forces the system into the

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non-linear regime. Two different forms of the excitation are included: (a) the first 8 seconds of El Centro to represent a distant event and, (b) a simulated earthquake shock to represent a close-in event. The system is assumed to be designed up to the limit of the linear range. This fixes the "yield point" of Figure 4. An increase in the excitation above the design basis event can be expected to cause yielding.

Table 1 lists the maximum response for the bi-linear and fully plastic system; the non-linear response is referenced to the linear design basis response. Subjecting the system to twice the design basis causes a non-linear response of the equipment. If the system had remained linear, the accelerations of the secondary mass would be twice the design basis. Except for the bi-linear system at frequencies \( \gamma > 1.5 \), the forces on the equipment are noticeably reduced by the non-linearities. This is the generally expected and beneficial result. But, by noting the ratio of non-linear displacements, the detrimental effect of non-linearities -- very high excursions into the non-linear range are evident. The worst response is for fully-plastic systems at high frequencies. The explanation for this can be seen by studying the time history plots of the base excitation of the equipment and its relative response (Figure 5). At certain times and for certain time intervals the impressed base accelerations simply exceed the resisting force available. In a static sense, the system has 'failed', continuing to accelerate as long as the force exceeds the resistance. The situation is arrested when the base excitation reverses. However, continued cycling of base acceleration causes a pronounced ratcheting effect.

In Figure 5 the time history of the equipment displacement \( \delta \) is plotted for the plastic system \((U=1.0)\) and \(r=2.0\), that is system 8 of Table 1. The yield displacement is 0.241; the actual maximum displacement is 5.823; an excursion into the yield zone which would demand a ductility factor exceeding 20. Figure 6 is the time history response of system 4; the value \(r=1.00\) means that the equipment, in the linear mode, is in resonance with the supporting structure. Figure 7 traces the force deflection curve for time \(1.75 < t < 3.5\) seconds. It is seen that even though the secondary mass response is substantially non-linear it does not "escape" resonance.

**Near Field Excitation:** When the excitation \( \delta \) is changed, the response changes. Using an artificial record to represent a near field excitation the response shown in Figure 8 was obtained. The scale of the "earthquake" was chosen so that the peak amplitude was twice that of the DBE. The structural parameters are the same as for Figure 5; that is, \(r=2.0, (U=1.0)\). The difference in response is clear. With the short duration, near field earthquake, only a ductility factor of 4 is required to maintain structural integrity.

**Impact:** Non-linear response does not necessarily mean a softening restoring function. Once equipment is dynamically disturbed it may pass through a clearance and impact either other equipment or the main supporting structure. Figure 9 shows the acceleration response of the secondary mass as it impacts the main mass. It was assumed that the gap was equal to 4 times the yield deflection, and that the impact was "cushioned" by a linear spring and viscous damper which was 10 times stiffer than the equipment supports; other coefficients are the same as for Figure 5. Of interest are the high acceleration spikes superimposed upon what would otherwise have been a lessened acceleration time history due to hysteretic, softening supports.

**Non-linear (Hysteretic) Effects of the Supporting Structure:** Table 2 shows the results of the analysis with a non-linear (hysteretic) support structure and linear equipment response.
As expected, twice the design basis excitation does not cause problems with the main structure if it has been designed for the appropriate ductility. In this case the ductility factor required would be in the order of 3. The response of the secondary mass is generally well below what would be expected for a straight linear extrapolation except for the lower frequencies. Thus a softening main structure will tend to protect rigid equipment but in turn expose the more flexible sub-systems to a higher acceleration field.

5. Element 3 - The Consequence

It is important to bring the Canadian seismic threat into context. Historically, deaths from seismic events have been extremely low. The only deaths recorded were for the 1929 earthquake on the Grand Banks, Newfoundland, where the resulting tsunami drowned 27 people in Placentia Bay, Newfoundland. When related to typical accidental death statistics, earthquake, as an initiator of death, rates low. For Canada, as well as in the U.S., the historical individual rate is below $1.0 \times 10^{-7}$ deaths per million population per year. The potential for enormous casualties, however, does exist. Many of the historic earthquakes could have had the potential for unparalleled disasters if they had occurred at different times and places. The "near-miss" aspect has been recognized and graphs of risk evaluations of earthquakes are characterized by flat frequency/casualty curves.

The population surrounding the site of a Nuclear Power Station is subject to a seismic risk that is quite independent of the station. In addition to this base seismic risk there is the additional risk of a man-made structure - the station - being damaged and adding to the consequences. Loss of life is usually associated with the failure of man-built structures or with industrial activities. Rural and domestic construction in Canada is such that there is very little likelihood of death in even the most severe earthquake. For example, the M7.2 earthquake near the mouth of the Saguenay River in 1925 caused no fatalities.

To obtain some feel for the number of casualties involved in a seismic event a computer simulation of a hypothetical province has been carried out. The province is assumed to have one nuclear reactor and a population of 4,000,000 people. The people are uniformly distributed; the seismic event is assumed to have an equal probability of striking anywhere in the province. The earthquake characteristics are as given in reference (5) and the maximum magnitude is 7. Uncontrolled failure of the plant is assumed at site accelerations 2.0 x DBE. The results indicate that on the average an individual within the province has an annual probability of death of $2 \times 10^{-7}$ due to the hypothetical background seismic events, and his chance of death due to a seismically-caused plant failure is about 1/10 of this value. However, for the population close to the reactor (<50 km) the individual probability increases to $2 \times 10^{-5}$. This rather simple simulation exercise illustrates 3 points:

(a) an individual close to a plant is at greater risk (perhaps significantly);
(b) the social seismic risk for a large body of people, say a province, is not severely affected by one reactor;
(c) seismic risks, both background and plant-caused, are still a very small part of total social risk of accidental death.

It has been suggested by others that the risk of a major seismically induced accident might be tolerable at frequency levels greater than the target figure for internal events ($10^{-7}$ events/annum), if it could be shown that this would constitute only a minor addition to the other non-nuclear damage that might be expected in the community as a result of the earthquake. This statement is not tenable and is not acceptable to the authors for two
reasons. First, such a policy would lead to the absurd conclusion that a reactor sited in a rural setting would have to be designed to more stringent requirements than one sited in an urban setting. Second, it is strongly felt that there are no technical barriers to constructing a plant that will meet the AECB consequence limits given the realistic seismic design methodology addressed herein. To reinforce this last statement, it should be noted that the preliminary investigation in which the AECB has participated for a Canadian Nuclear Icebreaker indicates that power plants for such ships can perform in a dynamic environment, not unlike a seismic environment, at values exceeding by an order of magnitude currently used values for land-based reactors.

Closure

It is difficult to treat seismic loading environment identical to other loading environments. Internal plant loading and most other external loading can be specified with reasonable confidence down to the very low frequency levels desirable for nuclear power plant safety. Unfortunately, seismic events have a limited historical data base and the basic source mechanism of earthquakes is poorly understood. It is however generally believed that there exists enough conservatism in the current seismic design methodology to ensure that the "performance standard" given in the form of published release limits is met. Current approaches world-wide are to quantify these seismic safety margins. This paper proposes an alternative way - a direct way. This direct approach requires explicit definition of an ensemble of maximum credible earthquake parameters, termed the Maximum Credible Earthquake and the safety provisions of the nuclear power plant must be shown, by realistic analysis and testing, to be adequate.

References

FIGURE 1
ANNUAL FREQUENCY (10^3) OF SITE ACCELERATION
pA (p=10^-g) | MAXIMUM MAGNITUDE 5.5(1), 6.0(+),
6.5(x), 7.0(-)

FIGURE 2
DISTANCE Y (km) VS SITE ACCELERATION X(g) FOR
SUB-SET OF 250,000 SEISMIC EVENTS >0.2 g

FIGURE 3
MODEL OF EQUIPMENT (M+E) AND
ITS SUPPORTING STRUCTURE (M)

FIGURE 4
BILINEAR FORCE (Y) VS DISPLACEMENT (X)
HYSTERETIC FUNCTION