Effect of High Frequency Ground Motion on the Seismic Design of Nuclear Power Plants

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INTRODUCTION

The seismic design of Canadian Nuclear Power Plants is based on CSA N289 series of standards on seismic qualification for structures, systems and equipment. (Ref.1). The design response spectra in the CSA standard is based on accelerogram records from western North America (WNA). However, recent strong motion recordings of ground accelerations suggest that accelerograms in eastern North America (ENA) exhibit significantly higher frequencies (>10 Hz) than those in WNA. Time history integration of the ENA records show that the CSA standard is exceeded at higher frequencies; this means that the design spectra which was intended to be a conservative representation of future seismic ground motion had under-estimated the potential for high frequency ground motion.

This paper reports on a series of seismic studies commissioned by the Atomic Energy Control Board of Canada (AECB). The drive behind this seismic program is to obtain the answer to three broad questions: a) what is the exceedance to the design spectra, b) what are the consequences of this exceedance, and c) what should be the direction for future seismic qualifications? The consequence of exceedance is explored in terms of observations of damage and non-damage in seismic events, review of shake table testing programs and analytical insights in terms of structural analysis models.

EXCEEDANCE TO THE DESIGN BASIS SPECTRA

Accelerograms recorded from six different earthquakes were selected. Table 1 lists the earthquakes, their magnitudes, M and epicentral distances, Δ. The horizontal components with the largest peak ground acceleration were used in the analyses. The time histories for these components are plotted in Figure 1. Response spectra at 2% damping for each of the accelerograms, normalized to 0.2 g peak acceleration, are presented in Figure 2. The exceedance to the CSA standard is sketched on the curve; above 15 Hz the design spectra is exceeded.

The Kern County & Parkfield earthquakes were chosen because they are "classics" of earthquake engineering and have played a dominant role in developing "standard" design spectra. These records are typical of those recorded in Western North America (WNA). Their lack of recorded high frequency data however, must be tempered by the fact that the bandwidth of the older instruments may have been significantly narrower than current instruments. We estimate that the Nyquist frequency f_n (maximum frequency which the data is capable of representing) for these two records is 25 Hz. For the other 4 earthquake records f_n is 50 Hz. For purposes of ENA-WNA comparisons, the pairs Nahanni-Parkfield and Kern County-Saguenay have similar epicentral distances; Parkfield and Saguenay have similar magnitudes.

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Table 1 SELECTED ACCELEROGRAMS

Event	Type	Date	Station	Component	Δ(km)	M
Kern County	WNA	21/7/1952	Taft Lincoln Tunnel	S69E	40	7.2
Parkfield	WNA	27/6/1966	Cholame Shandon # 5	N85E	5	6.1
Miramichi	ENA	31/3/1982	Loggie Lodge	S81E	6	4.0
Nahanni	?	23/12/1985	Site 1	N10E	7	6.8
Leroy, Ohio	ENA	31/1/1986	Perry Plant	N-S	17	4.8
Saguenay	ENA	25/11/1988	St. Andre	N-S	64	5.9

The Nahanni event is the largest ground motion ever recorded in Canada. It is somewhat of an anomaly, as it combines both the low frequency characteristics of WNA earthquakes and the high frequencies typical of ENA earthquakes, it is not yet clear if this is a typical ENA or WNA style earthquake. The Leroy, Ohio earthquake exceeded the operating limits of the Perry Nuclear Power Plant resulting in a delay in plant start-up, until it could be established that no damage had occurred. In fact, no damage was observed or indicated and the plant finally began full operation about 2 months after the earthquake. The Leroy earthquake is similar to Miramichi in terms of magnitude and the generation of high-frequency ground motion; it is considered to be a typical ENA event.

OBSERVATIONS OF DAMAGE AND NON-DAMAGE IN EPICENTRAL AREAS

For the 3 ENA earthquakes little damage of significance occurred for the Miramichi and Leroy events. For Miramichi it must be noted that the main shock (January 9, 1982 magnitude 5.5 to 5.9) was not recorded; the event shown was the strongest recorded in the after-shock sequences. The nearest industrial facility was the Heath Steels mines (70 km from epicentre). The ground shaking was met with considerable alarm as the mine was preparing for a major blast; there was concern that the charges had gone off prematurely. At the Brunswick Mining copper mine (100 km from epicentre) no damage occurred but the noise and vibration convinced the underground crews to take the prudent course and exit the mine. At an electric power station (120 km from the epicentre) several alarms went off. Even though the epicentre was in a wilderness area, a number of well-appointed summer and hunting lodges were in the area as well as logging roads and bridges. None of the structures, including some substantial stone chimneys, and none of the contents and furnishings showed any distortion and movement.

For the Saguenay event the damage reports were mixed. At the Alcan Smelter & Chemical Division Works located about 40 km from the epicentre no structural damage occurred but power failures caused considerable operational concerns that electrolytic cells would freeze. About 7 to 8 tons of HF was lost from a storage tank due to opening of a relief valve. Alcan operates a 2000 MW hydro-electric generating system extending up to 200 km from Chicoutimi; a number of generating units tripped due to relay chatter. The re-start of the generators took over 3½ hours. The most significant engineering damage occurred at the Bersimis #2 Hydro Electric Power Station where two sets of porcelain columns supporting breakers in the switchyard collapsed and 6 other sets were damaged. Damage was estimated at $800,000. This site was 200 km from the epicentre.

The Canadian damage experience, or better described as lack of damage, compares with similar experience of the behaviour of industrial facilities in California, Mexico and Chili. For example the Coalinga earthquake of May 2, 1983, (Ref. 2) magnitude 6.7, was centred in a large oil field that included numerous petro-chemical and other industrial and power installations. Except for some oil storage tank failures the effect of the earthquake on industrial structures and equipment was to cause some slight shifting or misalignment.

The results of the last 12 years of engineering investigations of damage and non-damage in epicentral regions has shown that little damage occurs to heavy industrial facilities sited on competent foundation material. In particular damage due to high frequency ground motion appears to be minimal.

REVIEW OF SHAKE TABLE TEST RECORDS

In the Canadian Nuclear Industry, equipment that is classified as DBE "B" such as instrumentation, valves, solenoids, electrical switchgear, is qualified by shake table testing (DBE refers to the design basis earthquake). To meet the qualification requirement the laboratory intends to provide a dynamic environment which exceeds the Required Response Spectrum (RRS).

We examined the records of shake table tests and have concluded that the vast majority of equipment so tested is subjected to extraordinarily severe levels of motion which are higher than the design basis ground motion or floor response spectra and are not likely to be surpassed during an earthquake. The testing laboratories normally limit their input frequencies to < 33 Hz; however, analysis of the table motion shows strong input frequencies above the intended frequencies. Those high frequencies occur due to impact through mechanical clearances of the activating rams and stabilising mechanism of the shake table. For one major testing laboratory the actual computed test response spectra (TRS) of the table show response exceeding 10 g (computed at 5% damping) in the frequency range up to and exceeding 100 Hz.

INSIGHTS FROM STRUCTURAL ANALYSIS MODELS

The purpose of the analytical models is to: a) determine the inertia forces attracted by the structure due to ground motion, b) to determine the response or level of damage these forces are causing and c) to rule on the acceptability of this response or damage. In practice, the models range from coefficient type models used in building codes, to eigenvalue-eigenvector extraction techniques of large (10^3-10^4 degrees of freedom) finite element models, through to full non-linear, time-history simulation models representing translational and torsional degrees of freedom. A second level of models may also be added to simulate seismic wave foundation interaction. Table 2 lists the models that we reviewed for the purpose of assessing the consequences of high frequency motion.

Table 2 STRUCTURAL MECHANICS MODELS

MODEL	MEASURE OF SEISMIC LOADING	ACCEPTANCE CRITERIA	REFERENCE
1	$\max_{t} \lvert a \rvert$	stress $\sigma \le \sigma_y$	Ref (1) CSA N289
2	$\max_{t} \lvert v \rvert$	distortion $x \le \mu x_y$	Ref (3) NBCC
3	$\int_0^T \lvert a(t) \rvert dt$	integral ≤ threshold	Ref (4) EPRI
4	$m \frac{d^2(x+u)}{dt^2} = -f(\dot{x}, x, t)$	trace $\{f(\dot{x}, x, t)$ vs $x(t)\}$ ≤ experimentally verified trace	Ref (5)

Model 1 is an abstract of the nuclear code CSA N289 approach; force levels are based on peak spectral accelerations and stress is to be maintained below the yield point. Model 2 is used by the National Building Code of Canada. Starting from 1980 the NBCC switched from an acceleration force determination approach to one based on maximum ground velocity on the argument that the integral of ground acceleration $\int_0^t a(t)\,dt$ was a better indicator of potential ground damage. Model 3 was developed by EPRI to suggest an alternative to existing measures of OBE exceedance. Current USNRC regulations require a plant shutdown if the OBE design response spectra is exceeded. Non damaging high frequency ground motion with OBE exceedance have occurred at the Virgil C. Summer plant in South Carolina 1978 and 1979 and at the Perry plant in Ohio in 1986. No such regulation exists in Canada. The integral $\int_0^t |a(t)|\,dt$ is used to establish a high-frequency cutoff on the response spectrum. If the integral is $\leq$ a numerical threshold then the spectra above 10 Hz can be ignored. The last model, model 4, represent direct integration of the equation of motion with the function $f(\dot{x},x,t)$ representing the restoring force due to the deformation and relative velocity of the responding structure. The function 'f' models non-linear stick-slip or hysteretic material properties. No simple acceptance criteria exist for non-linear analysis; in general the trace of the response in terms of velocity reversals, number of hysteretic cycles, internal energy consumed, is compared against similar experimental traces. A qualitative judgement is then made as to the acceptability of the analysis.

The models increase in complexity and are truer representation of ductile behaviour as the numbers increase from 1 to 4. Model 1 is a realistic description of brittle (glass-like with linear σ/ε curves) materials that fracture at σ_y (example is Bersimis #2). Stress is directly proportional to acceleration and for this class of materials peak acceleration is an appropriate measure of damage.

The vast majority of structural systems used in the construction of nuclear power plants are capable of ductile behaviour either through the property of the engineering materials used or through the connection details. Structural systems can dissipate considerable energy in non-linear deformation. In a seismic environment the loads are inertia caused and transient. Damage will occur if: a) the internal distortion becomes significant (eg. exceeds the ductility limit, impacts other structures, or causes lack of stability) and b) the build-up of kinetic energy exceeds the ability of the structural system to dissipate this energy (through viscous or hysteretic damping, friction at connections, or by sliding). For ductile systems where an energy absorption capability exists after a design stress has been exceeded the build-up seismic energy may be accommodated by extremely small movements. An example based on model #4 follows.

Calculations based on a single degree of freedom model of mass 2 metric tonnes supported to have a natural frequency of 36 Hz and subjected to the Miramichi (Loggie Lodge) ground motion will be subjected to a 10,000 N force (0.5g). Stresses in the steel support is computed at 200 MPa (near yield) but the kinetic energy due to the velocity of the ground motion is less than 2.0 N·m. While the stress computed is high the 2.0 N·m can easily be absorbed by the slightest distortion of the supports.

EXCEEDANCE OF DESIGN BASIS SPECTRA AT HIGH FREQUENCIES

The major structural modes for the reactor and auxiliary buildings have fundamental frequencies in the range of 2 to 8 Hz. These structures will not respond significantly in the higher modes; it can be expected that high input frequencies will be attenuated or filtered by the "softer" structural modes. However, a certain class of equipment that supports emergency functions of the reactor, that must work during and following the seismic ground motion, is installed in one story buildings outside the main reactor building complex or near the base mats. These smaller structures and the equipment within them will experience the high frequency excitation similar to the sensors that originally recorded them. For equipment or structures qualified by analysis the higher accelerations would result in higher calculated stresses; there is a possibility that the analysis will not "pass" the maximum stress criteria.

For equipment that has been shake table tested our review of test records shows that such equipment has, in general, been severely tested at frequencies up to and far above 33 Hz; there is little concern that the higher frequencies found in recent Canadian earthquakes have not been simulated in currently installed equipment.

DISCUSSION, CONCLUSIONS & RECOMMENDATIONS

For nuclear power reactors the approach used by CSA-N289 was to chose a conservative inertial loading apportionment and damage criteria. The design approach is linearly elastic with a dynamic analysis approach required for all structures and equipment up to 33 Hz and a static analysis based on peak acceleration for values > 33 Hz. All analytical criteria are eventually calibrated against performance in real seismic events. Observations in many recent earthquakes (≈ 70 documented events) have shown that no significant damage has occurred to rigid structures and equipment whether or not such items have been specifically designed to resist seismic forces. Structural analysis models can demonstrate the reason why such damage should be small and we have provided an illustrative example. The design criteria adopted in CSA N289 are conservative. Such conservative criteria are appropriate for: a) ensuring leak-tightness of containment structures and pressure-retaining components and, b) control of deflections in rotating components such as pumps and motors. However, for a large class of structures and equipment (eg. structural floors and platforms, bracing, cable trays, instrumentation racks and cabinets) more permissive models may be appropriate.

It is our conclusion that the exceedance to the design basis spectra in the high frequency range observed in recent ENA earthquakes is off-set by the considerable conservatism inherent in the design code. Yet, we conclude that it would be preferable to analyze and document the response to high frequency, high acceleration ground motion explicity. We recommend that studies be undertaken on how best to augment the analysis sections of the CSA-N289 seismic code to include: a) non-linear analysis models and acceptance criteria, b) acceptable ductility ratios of structural systems (eg. bolted & welded structured steels, reinforced and prestressed concrete, anchors) and c) allowance for the benefits of strain rate on concrete material properties.

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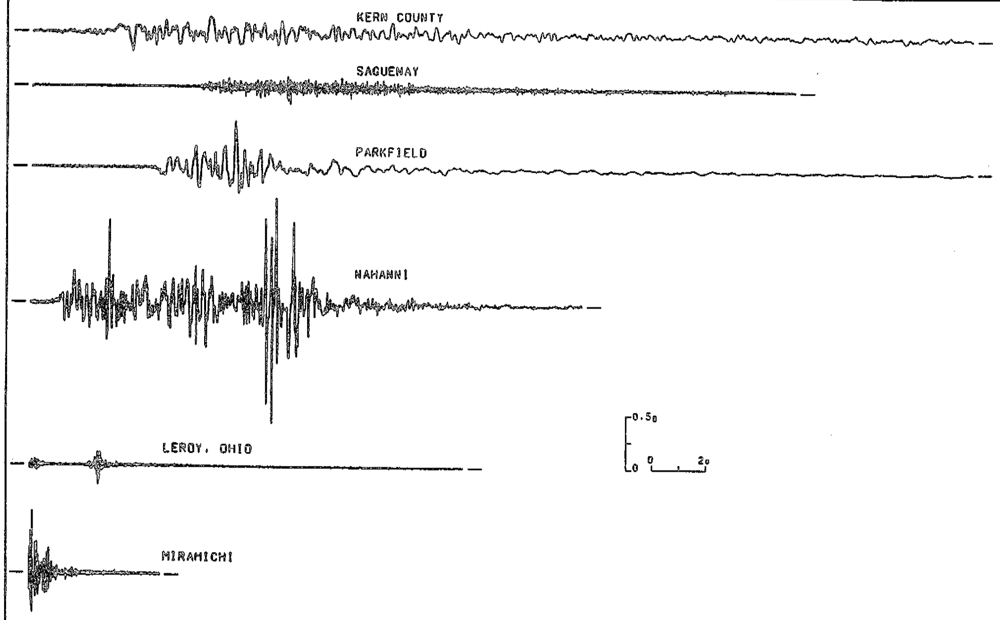

Fig. 1 Selected Accelerograms

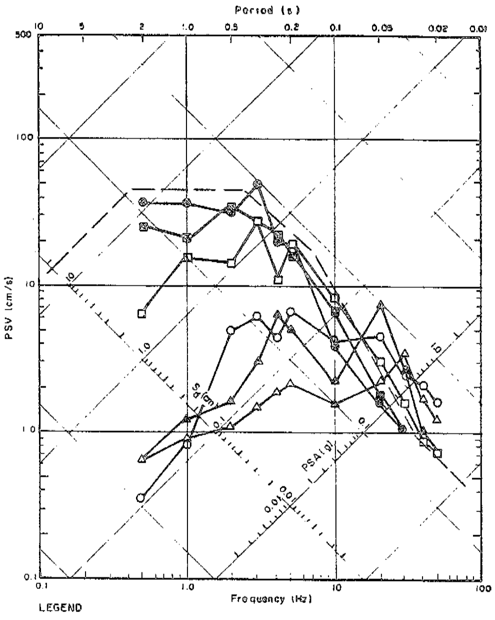

Fig. 2 Response Spectra of Scaled Selected Accelerograms (2% Damping)

[--- Superimposed Design Basis Earthquake Horizontal Ground Response Spectra]