SEISMIC RISK ANALYSIS FOR
THE WESTINGHOUSE ELECTRIC FACILITY
CHESWICK, PENNSYLVANIA

P.O. 66-61003

Submitted to

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A THREE DIMENSIONAL PLOT OF
HISTORICAL SEISMICITY AROUND THE SITE

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1.0 INTRODUCTION AND SUMMARY

In this report, TERA Corporation presents the results of a detailed seismic risk analysis of the Westinghouse Electric plutonium fuel development facility at Cheswick, Pennsylvania.

This report is one part of a larger effort being directed by the U.S. Nuclear Regulatory Commission. The NRC's objective in commissioning the overall report is to assess and improve, to the extent practicable, the ability of this facility to withstand adverse natural phenomena without loss of capacity. This report focuses on earthquakes; the other natural hazards, being addressed in separate reports, are severe weather (strong winds and tornados) and floods. The overall report will provide an assessment of the consequences of an accident resulting from any of these natural phenomena. The assessment will express a quantitative probabilistic measure of the potential structural damage and the release function. It will also provide a probabilistic estimate of the resulting dose of radioactivity to the public.

To ensure credible results, very sophisticated but well accepted techniques were employed in the analysis. The calculational method we used, which is based on Cornell's work (1968), has been previously applied to safety evaluations of major projects.

The historical seismic record was established after a review of available literature, consultation with operators of local seismic arrays and examination of appropriate seismic data bases including the USGS, LASA, NOAA, USC and GS, and NEIS bases.

Because of the aseismicity of the region around the site, an analysis different from the conventional closest approach in a tectonic province was adapted. Earthquakes as far from the site as 1,000 km were included, as were the possibility of earthquakes at the site. In addition, various uncertainties in the input were explicitly considered in the analysis. For example, allowance was made for

both the uncertainty in predicting maximum possible earthquakes in the region and the effect of the dispersion of data about the best fit attenuation relation.

The attenuation relationship we applied is, we feel, the best available. It is derived from two of the most recent, advanced studies relating earthquake intensity reports and acceleration and is unique in that

- It incorporates a thorough analysis of the effects from the 1886 Charleston, South Carolina Earthquake;
- It is based on a recent analysis of almost 1500 world-wide strong motion records; and
- It is consistent with the newly available strong motion acceleration data for the Eastern-Central United States.

Finally, and most important, the project has benefited from significant contributions from, and final review by, three seismologists with particular expertise in the local and regional seismology:

Professor Shelton Alexander, Pennsylvania State University
Professor G.A. Bollinger, Virginia Polytechnic Institute and State University
Professor R. Herrmann, St. Louis University

The results of our risk analysis, which include a Bayesian estimate of the uncertainties, are presented in Figure 1.1 expressed as return period accelerations. The best estimate curve indicates that the Westinghouse facility will experience 0.05 g every 220 years and 0.10 g every 1400 years. The bounding curves roughly represent the one standard deviation confidence limits about our best estimate, reflecting the uncertainty in certain of the input. Detailed examination of the results show that the accelerations are very insensitive to the details of the source region geometries or the historical earthquake statistics in each region and that each of the source regions contributes almost equally to the cumulative risk at the site.

Figure 1.1

RETURN PERIOD - PEAK ACCELERATIONS
FOR THE WESTINGHOUSE FACILITY

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If required for structural analysis, acceleration response spectra for the site can be constructed by scaling the mean response spectrum for alluvium in WASH 1255 by these peak accelerations.

2.0 SEISMIC RISK METHODOLOGY

A seismic risk analysis is only as credible as the risk analysis methodology and the input to it. This section presents the basis for our selection of a probabilistic Poisson model for the risk assessment at the Westinghouse facility.

There are generally two distinctly different approaches to seismic risk analysis; probabilistic and deterministic.

Using the deterministic approach, the analyst judgmentally decides that an earthquake of a given magnitude or intensity occurs at a specific location. He then attenuates the ground motion from the earthquake source to the site and determines the effects of that quake. The problem in using this approach is that it is difficult to define the margin of safety or the degree of conservatism in the resulting design parameters. Analysts are often asked to provide information on the "maximum possible" or "most probable" earthquakes for design purposes, but the deterministic approach does not easily provide those answers.

A probabilistic approach, on the other hand, quantifies the uncertainty in the number, size, and location of possible future earthquakes and allows an analyst to present the trade-off between more costly designs or retrofits and the economic or social impact of a failure. Because the product of a probabilistic approach is a measure of the seismic risk expressed in terms of return period, this trade-off can easily be quantified.

Although the probabilistic approach requires significantly more effort than the deterministic approach, it has the following advantages:

- It quantifies the risk in terms of return period;
- It rigorously incorporates the complete historical seismic record;
- It can incorporate the judgment and experience of the analyst;

- It accounts for incomplete knowledge regarding the location of faults;
- It has the flexibility to assess the risk at the site in terms of spectral acceleration, velocity, displacement, or earthquake intensity.

The method is particularly appropriate for the Westinghouse facility for two reasons. First, as will be shown below, the Pittsburgh area is very aseismic and it would therefore be very difficult, using conventional deterministic methods, to establish a design earthquake magnitude. Second, the seismicity of eastern United States is very diffuse and it cannot be correlated with surface faulting as it can be in the western United States. The location of the design earthquakes in the eastern United States is therefore particularly uncertain. The strength of the probabilistic approach is its ability to quantify these uncertainties.

The credibility of the probabilistic approach has been established through detailed technical review of its application to several important projects and areas. Recent applications include assessments of the seismic risk in Boston (Cornell, 1974), in the San Francisco Bay Area (Vagliente, 1973), in the Puget Sound Area (Stepp, 1971) and continental United States (Algermissen and Perkins, 1976). Results of these studies have been applied to, among other areas:

- Development of long-range earthquake engineering research goals;
- Planning decisions for urban development;
- Environmental hazards associated with the milling of uranium; and
- Design considerations for radioactive waste repositories.

This diversity of application demonstrates the inherent flexibility of the risk assessment approach.

THEORY

The risk calculations can be fundamentally represented by the total probability theorem

$$P[A] = \iint P[A/m \text{ and } r] f_M(m) f_R(r) dm dr$$

where P indicates probability, A is the event whose probability is sought, and M and R are continuous, independent random variables which influence A . The probability that A will occur can be calculated by multiplying the conditional probability of A , given events m and r , times the probabilities of m and r , and integrating over all possible values of m and r .

In our assessment of the Westinghouse facility, A will be taken as maximum acceleration and therefore

$$P[A/m \text{ and } r]$$

will be derived from data relating peak acceleration to epicentral distance and earthquake magnitude. Often known as attenuation data, these data are usually lognormally distributed around a mean relationship of the form (McGuire, 1977a).

$$A = C_1 e^{C_2 M} (R+r_0)^{C_3}$$

The distribution on earthquake magnitude, $f_M(m)$, can readily be derived from an actual or postulated frequency relationship of the form

$$\log N = a - bM$$

where N is the number of earthquakes having magnitude greater than M , and a and b are constants characteristic of the particular source region under consideration. It follows (Cornell, 1968) that f_M can be derived from the cumulative distribution function, F_M , which has the form,

$$F_M = k (1 - e^{-\beta M})$$

where k is a normalizing constant and $\beta = b \ln 10$.

The distribution on distance, $f_R(r)$, depends on the geometry of the problem under consideration. For simple geometries, the distributions can often be integrated analytically. Realistic geometries however, require numerical evaluation of the integral. A very versatile computer program has been developed (McGuire, 1976b) that incorporates the theory presented above with a numerical integration scheme that allows for evaluation of very complex source-site geometries. The theory of seismic risk assessment by this approach is outlined below.

First, the historical earthquake record and local attenuation data are combined with the experience of the analyst to produce the functional relationships applicable to the area under consideration. The source regions are divided into circular sectors and proportional seismicity is allocated to each sector. The total expected number of events causing maximum accelerations at the site greater than a particular test acceleration are obtained by summing the events from each sector within each source region. The risk associated with this test acceleration is then calculated under the conventional assumption that earthquakes have a Poisson distribution in time. It then follows that the return period is simply the reciprocal of the risk.

3.0 GEOLOGY

SITE DESCRIPTION

Location

The Westinghouse site is situated in the township of Harmar, Pennsylvania, approximately 10 miles northeast of the city of Pittsburgh. This site is located at latitude $40^{\circ}32'58''$ N and longitude $79^{\circ}48'45''$ W within the New Kensington West Quadrangle (U.S.G.S. 7.5' topographic map series).

Topography

The Harmar area lies within the east central part of the Allegheny Plateau physiographic province. The plateau in this area has been thoroughly dissected into narrow accordant ridges, and most of the land surface is moderately to steeply sloping. Maximum relief between remnants of the plateau surface is approximately 550 feet. Local relief in the smaller valleys of the plateau usually ranges between 300 and 400 feet. Valley side slopes are commonly abrupt and steep with average slope angles ranging between 15 and 30 degrees.

The Westinghouse site is situated on a large remnant of a broad alluvial terrace within the valley of the Allegheny River. The elevation of this terrace ranges between 960 and 980 feet, approximately 220 feet above the level of the Allegheny River floodplain. This high terrace is dissected by several short, high gradient streams that have cut narrow, steep-sided valleys as much as 150 feet into the terrace surface. The Westinghouse site is located along the outer edge of the terrace and a narrow, steep-sided valley approximately 140 feet deep flanks the site's western edge.

Surficial Geology

The Westinghouse site is underlain by unconsolidated fluvial sands, gravels and occasional layered clays of probable Illinoian age (Wagner and others, 1970,

1975a; O'Neill, 1974). These sediments are deeply weathered and contain numerous erratic gravels of widely varying lithology. Borehole logs and visual inspection reveal that these deposits are almost entirely composed of moderately dense, brown gravelly sands with occasional lenses and thin layers of brown sandy to silty clay and clayey silt. Both bedrock outcrops and borehole logs in the vicinity of the site indicate that these gravels form a veneer 35 to 40 feet thick over what is probably a buried strath terrace surface.

These Illinoian terrace deposits are considered to be a good source of relatively high quality sand (O'Neill, 1974). Indeed, the Westinghouse site is bordered on its east, south and west sides by inactive sand and gravel pits.

Soils developed on the sand and gravel deposits of this terrace have been mapped as Rainsboro soils by the U.S. Soil Conservation Service (1973a). These soils are typically deep and moderately well drained but have a relatively low permeability. According to the Soil Conservation Service (1973b), these soils are characterized by a high seasonal water table and significant frost susceptibility. However, the upper 5 to 10 feet of the natural land surface in the area of the Westinghouse site was removed prior to construction of plant facilities so that the present land surface of the site is relatively permeable and well drained.

Bedrock Geology

The terrace capping gravels of the Westinghouse site are underlain by a 16,000 foot sequence of Paleozoic sedimentary rocks. These rocks are nearly flat-lying, however, and only the top few hundred feet of these strata are relevant to the present study. The uppermost 250 to 300 feet of this Paleozoic sequence are Pennsylvanian shales and sandstones of the Glenshaw Formation. These rocks are underlain by the Upper Freeport coal, a thick, laterally continuous seam that comprises the bulk of mineable coal reserves in the Pittsburgh area. This seam defines the top of the Allegheny Group, approximately 300 feet of interbedded shale, claystone, sandstone, limestone and coal (Richardson, 1932; Wagner and others, 1970, 1975a).

Bedrock Structure

The bedrock strata underlying the region surrounding the Westinghouse site are nearly flat-lying. Bedrock dips are almost everywhere less than 50 feet per mile (0.5°) and dips of 10 to 20 feet per mile are most common. In the immediate vicinity of the site, Pomeroy and Davies (1975) have mapped a general northward dip of approximately 40 feet per mile into the north side-slope of the Allegheny River Valley. The axis of a gentle northeast-southwest trending anticline (the Amity anticline) is located about one mile southeast of the site, but the amplitude of this five to six mile wide structure is only 70 to 80 feet (Wagner and others, 1975b).

No faults are indicated on any geologic maps or in any other published literature describing the area. However, the drainage pattern of the region surrounding the site contains a strong angulate component indicating significant control of erosion and drainage development by jointing or other bedrock fracturing. Many stream valleys are strikingly linear and, although a detailed morphologic analysis of these valleys has not been made, generally northwest-southeast and northeast-southwest trends predominate. Gold and others (1974b) mapped a series of megalineaments throughout western Pennsylvania on the basis of interpretations of Landsat imagery. The orientations of these megalineaments and the preferred orientations of the linear valleys of the region surrounding the site are similar and may be generically related.

To better understand the megalineaments and the angulate drainage of the region and to investigate their possible interrelationships, a detailed analysis of these features should be undertaken. This relatively modest analysis should include:

- Identification and mapping of the linear valleys; and
- Measurement of the lengths and orientations of these valleys from analyses of topographic maps and small scale (1/60,000) vertical aerial photographs.

Valley orientations should then be statistically analyzed and correlated with the orientations of the Landsat megalineaments. Field investigations of these features might also be attempted, but in view of the nature of these features and the poor quality of outcrops in the region, such work would be very time consuming and might be unproductive.

EVALUATION OF NON-TECTONIC GEOLOGIC HAZARDS

A preliminary evaluation of the non-tectonic geologic hazards associated with the Westinghouse site was made on the basis of:

- A thorough search for and review of existing maps, reports and other published and unpublished geologic data describing the region surrounding the site; and
- A field examination of the geomorphic and surficial geologic characteristics of the site on 13 May 1977.

Aerial photographic stereo coverage of the site area could not be located during the site visit.

Flooding

The Westinghouse site is situated on a Pleistocene terrace, approximately 220 feet above the floodplain of the Allegheny River; and small, high gradient streams have cut narrow valleys as much as 150 feet deep into the surface of this terrace. Therefore, it is apparent that the Westinghouse site can not be inundated by any credible flood.

Slope Failure

The Westinghouse site is located 600 to 800 feet from the edge of a sand and gravel capped terrace high above the floodplain of the Allegheny River. In the vicinity of the site, the scarp of this terrace has an average slope of 8° and an average height of almost 180 feet. The upper 20 to 30 feet of this scarp has been

flattened by quarrying of the terrace capping alluvium and the lower slopes of the scarp are supported by gently northward dipping shales and sandstones of the Glenshaw Formation. The moderate slope, moderately strong materials and stable structure of this scarp combine to make it as stable as any slope in the area.

While perhaps increasing the stability of the main terrace scarp, quarrying of the terrace capping sands and gravels in the immediate vicinity of the site has lessened the stability of the site itself, although probably not significantly. The quarrying approach approximately 150 feet of the site along its southwest and west flanks. Along the southwest side of the site, the quarry scarp is about 30 feet high and has a smooth linear slope of approximately 25° . On the west of the site, this scarp steepens and lengthens near a narrow, steep-sided stream valley approximately 140 feet deep. A small arcuate slump approximately 30 feet wide and 20 to 40 feet long has occurred along this steeper, higher scarp. This slump is located about 150 feet from the site's outer perimeter fence and is probably the result of a poorly designed drainage pipe associated with the site. This slump almost certainly does not pose any significant threat to the integrity of the site at the present time, but corrective action should be taken to prevent additional slumping and headward expansion of the slump scarp. In addition, the proximity of this high, steep free face in unconsolidated material is significant to the seismic evaluation of this site.

Differential Surface Subsidence

The Westinghouse site is underlain by the Upper Freeport Coal at a depth of approximately 300 feet (Wagner and others, 1975a; Craft and others, 1976). This seam, 4 to 7 feet thick in the site area, has been extensively mined to within a few hundred feet of the site's northern edge (Bushnell and Peak, 1975; Davies and others, 1976) and the potential of differential surface subsidence on the site as a result of the undermining of this adjacent area may be significant.

To precisely assess the significance of this possible hazard, the following actions should be taken:

- The location and extent of undermining should be estimated as precisely as possible by an exhaustive search and analysis of existing mine records, perhaps confirmed by a surface boring program; and
- Specialists intimately familiar with the problem of mine subsidence in the Pittsburgh area should be consulted to estimate the extent and severity of possible future mine subsidence in the site area.

Because of the limited number and poor exposure of bedrock outcrops in the site area, a boring program may be the only sure way to determine whether or not subsidence has already taken place in the site area.

4.0 SEISMOLOGY

While the detailed elements of the seismic risk assessment are discussed in Section 5.0, the historical seismic record is of such significance that it is discussed separately below.

A complete evaluation of the historical record is the keystone to the risk assessment because of the important time and spatial distribution information it contains. With regard to time, the record provides detailed historical earthquake frequency information that can best be represented by the relationship, $\log N = a - bM$. The spatial distribution of earthquakes around the site can often be used to delineate seismic source regions within which earthquakes have common characteristics.

Unfortunately, earthquakes have been reliably reported only since the 1930s when a nationwide earthquake instrumentation program was started. The pre-1930 record is a very valuable supplement to the recent recorded data but due to sparse settlement and scattered intensity reports, much of these data cannot be reliably used in developing earthquake statistics. Our general approach is to use the recent recorded data to determine the statistics for magnitude six and less earthquakes and to include the entire historical record in determination of statistics for larger earthquake magnitudes.

We have collected and integrated the data from several seismic data bases to ensure the most complete coverage. The primary source of data was the magnetic tape supplied by the U.S. Department of Commerce NOAA containing the earthquake history through 1975. The data for the site area were checked and extended to 1977 by comparing them with other independent data. The Alexandria Laboratories of Teledyne Geotech maintains a data base which consists of LASA, USGS, USC & GS and NEIS data, and this data provided the most complete check and extension. The data through 1974 were checked

against the USGS data, provided by D. Perkins. Finally, data obtained by Weston Geophysical Research, Inc. for the Beaver Valley PSAR were examined as a final check. Besides these direct comparisons, the availability of local unreported data was considered by checking with local experts:

Professor G.A. Bollinger
Virginia Polytechnic Institute and State University

Professor Shelton Alexander
Pennsylvania State University

Professor Walter Pilant
University of Pittsburgh

Dr. Arthur Socolow
Pennsylvania State Geologist

The resulting integrated data base for the relevant area around Pittsburgh is plotted in Figure 4.1. Note that the data base is specified in terms of earthquake magnitude; this follows from our emphasis on the recent recorded data, which is in terms of magnitude. When it is required to incorporate an estimate of the epicentral intensity in the data base, we use the Gutenberg-Richter relationship (1956)

$$M = 1.3 + 0.61_e$$

to determine the equivalent magnitude. The validity of this relation for the eastern United States has been confirmed by comparisons between it and the more current seismic data base (Chinnery and Rodgers, 1973).

There are several important features of the historical data that must be included in any risk assessment. The most important is the obvious clustering of seismicity in several localities.

The most apparent clustering is in the New Madrid area, site of the famous 1811-12 earthquakes. These earthquakes, which were the largest ever experienced in the eastern United States, resulted in intensities as high as V (M.M.) in the

Figure 4.1

HISTORICAL EARTHQUAKES IN THE VICINITY OF THE SITE

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Pittsburgh area. The shocks were felt as far away as Boston, Massachusetts, and the total felt area was by far the largest ever experienced on this continent. The area was well known for its seismicity even before the 1811-12 earthquakes, with historical accounts going back even into Indian legends. As can be seen from Figure 4.1, the seismicity around New Madrid has been relatively contained, thus suggesting a local tectonic origin.

Other areas of repetitive seismic activity include the Fairport-Cleveland, Ohio area, the Attica, New York area, and the Anna, Ohio area. The activity in the Fairport-Cleveland, Ohio area has been minor. The largest earthquakes associated with this area are of intensity V (M.M.). The Attica, New York area, 180 miles northeast of the site, experienced an earthquake of intensity VIII (M.M.) (August 12, 1929) and two earthquakes of intensity VI (M.M.). In the Anna, Ohio area, approximately 200 miles west of the site, one earthquake of intensity VII-VIII (M.M.) (March 9, 1937) and three earthquakes of intensity VII (M.M.) have occurred. Another important, although very distant, source region exists along the St. Lawrence Seaway. This region, which is one of the two most active regions in Canada, has experienced several earthquakes--most notably the February 28, 1925 St. Lawrence Earthquake.

Another striking feature of the historical record is the absence of seismic activity in western Pennsylvania. It is apparent that the historical seismic risk in the Pittsburgh area has been due to earthquakes originating on distant structures. For example, the Attica Earthquake August 12, 1929 was felt with intensity IV at Pittsburgh, the Anna, Ohio Earthquake of March 8, 1937 resulted in intensity II, and the Cornwall-Mossena Earthquake (St. Lawrence Seaway Region--September 4, 1944) was felt with intensity III. Finally, as mentioned above, the New Madrid Earthquakes resulted in local intensities around Pittsburgh as high as V. These are very low intensities compared to those experienced in surrounding regions and thus serves to further demonstrate the consequences of local aseismicity.

5.0 CALCULATIONS AND RESULTS

In the previous sections, we have described the regional seismicity around Pittsburgh and have discussed the most appropriate method of risk analysis. In this section, we apply these concepts to the Westinghouse site. The detailed input to the calculational model is described below, followed by a presentation of the results.

INPUT

As described in Section 2.0, Seismic Risk Methodology, the input to a probabilistic risk assessment comprises earthquake frequency relations, attenuation functions and a specification of local source regions. Because risk assessment calculations are very sensitive to the particular composition of the input, we consulted with several eminent seismologists during the preparation of input for the Westinghouse facility analysis. Major contributions in this effort were made first by Professor G. A. Bollinger, (Virginia Polytechnic Institute and State University), Professor R. Herrmann (St. Louis University), and later by Professor S. Alexander (Pennsylvania State University).

Source Regions

After a thorough review of the historical seismicity (Figure 5.1) and geologic/geophysical parameters such as gravity, magnetics, tectonics, and surface geology, it was agreed that the most appropriate source regions were those defined by Algermissen and Perkins (1976). Their definition of the source zones was based on the reasonable assumption that future earthquake occurrences will have the same general statistics as historical earthquakes and that the historical variation of earthquake statistics from region to region can be used to delimit general source regions. The final definition of the source region's boundaries was based on the average separation distance for earthquakes of the largest intensities. The representation of the source regions synthesized all the available historical seismicity data and the state of knowledge of the relationship between geologic structure and historical seismicity.

Figure 5.1
HISTORICAL EARTHQUAKES IN THE VICINITY OF THE SITE

Figure 5.2 shows the appropriate source regions for the Babcock and Wilcox facility. The final determination of the most appropriate set of regions evolved from sensitivity studies. The regions are generally those contained in a 500-kilometer radius around the site, and with the exception of region 61, those regions more distant from the site did not contribute to the seismic risk. Region 61, which is the source region for the New Madrid earthquakes, was included because it has the low probability capability for large earthquakes that could contribute to the risk at the site. We have made two minor modifications to the Algermissen and Perkins source regions. First, for consistency with current NRC policy, we have extended the New Madrid source region (region 61) to the north, and second, we have truncated region 68 such that its outer boundary is 500 kilometers from the site. The seismicity of that region is allocated accordingly.

Source Region Seismicity

Algermissen and Perkins (1976) calculated the rates at which earthquakes occur in each of their source regions based on the seismic data available at that time (1974). These rates, which are related to coefficients in the expression

$$\log N = a - bI$$

are presented in Table 5.1.

These parameters define the incremental distribution of earthquake magnitudes (that is the number of earthquakes between I and $I + dI$) up to an hypothesized maximum intensity. The Algermissen and Perkins (1976) maximum magnitudes corresponded to the largest historically observed earthquakes.

We updated this data base to 1977 as described in Section 4.0 and our re-examination of the seismicity in each region indicated that there was no change in the earthquake statistics and therefore no basis for altering the Algermissen and Perkins statistical parameters. The maximum magnitude earthquake is a

Figure 5.2

SEISMIC SOURCE REGIONS
USED IN THE ANALYSIS

(Source Region Numbers From
Algermissen and Perkins, 1976)

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Table 5.1
ALGERMISSEN AND PERKINS (1976)
EARTHQUAKE PARAMETERS

<u>Zone No.</u>	<u>Maximum Historical Earthquake</u>	<u>No. of MM Maximum Intensity V's per 100 Years</u>	<u>b</u>
61	X	84.5	.50
62	VIII	22.0	.50
63	VIII	22.1	.64
66	VIII	13.0	.59
67	VII	7.8	.59
68	VIII	69.1	.67

very uncertain parameter, particularly in the less seismic areas that we are considering here. Accordingly, we judged that it was reasonable and moderate to assume that each region was capable of earthquakes of roughly one-half magnitude unit greater than the largest magnitude earthquake in the historical record. The Gutenberg-Richter relationship is also used here to relate historical intensity to magnitude. To account for uncertainty in not only this relationship, but also the physical arguments leading to a specific maximum upper bound earthquake, we also consider in our calculations, described below, the possibility for upper bound cut-offs of plus and minus one-half magnitude unit about this maximum upper bound earthquake. For calculational simplicity, we also specify a lower cut-off magnitude for each region. Sensitivity studies showed that earthquakes with magnitudes less than the lower cut-off do not affect the risk at the site. Finally, the calculational risk model requires that the distribution of earthquakes be specified as a complementary cumulative distribution (number of earthquakes with magnitudes greater than M) rather than incrementally. The results of this integration, up to the upper cut-off magnitude, are presented in Table 5.2 along with the upper and lower cut-off magnitudes.

Note that Table 5.2 includes a background zone of seismicity; these parameters describe the earthquake parameters for the region in Figure 5.2 containing the site. Although this area, as noted by Algermissen and Perkins, is very aseismic, the inclusion of a reasonable seismicity model is essential for proper estimation of the risk within the region. The indicated parameters are based on the data and on the advice of Perkins (personal communication, July 7, 1977). As above, the upper cut-off corresponds to one magnitude unit greater than the largest historical earthquake.

Attenuation

The attenuation relationship was chosen for the credibility it has obtained from extensive review and evaluation. None of the other available relationships (McGuire, 1976b lists 25 published relations) has been reviewed or scrutinized as carefully as the components of the one used in this analysis.

Table 5.2
EARTHQUAKE PARAMETERS USED IN THE RISK CALCULATIONS

Zone No.	Lower Cut-off Magnitude	Upper Cut-off Magnitude	Number of earthquakes greater than lower cut-off magnitude per year	b
61	4.0	8.0	2.24	0.50
62	4.0	6.5	.59	0.50
63	4.0	6.5	.63	0.64
66	4.0	6.5	.36	0.59
67	4.0	6.0	.21	0.59
68	4.0	6.5	.70	0.67
Back- ground	5.0	5.5	.004	0.70

The basic approach is to develop the functional form of the relationship by synthesizing the results of several previous investigations. The specific relationship is then defined by a fit of the resulting functional form to the only available strong motion data in Central/Eastern United States.

The functional form is developed from three separate regression analyses. The analyses resulted in best fits to the data for:

- Site intensity vs. distance and epicentral intensity
- Peak acceleration vs. site intensity, earthquake magnitude and distance
- Earthquake magnitude vs. epicentral intensity

The first of these relationships is contained in Professor Bollinger's contribution to USGS Professional Paper 1028 on the Charleston, South Carolina, 1886 earthquake (Bollinger, 1977). Bollinger's analysis of the 800 intensity observations from that earthquake resulted in the development of a new intensity attenuation relation that is similar to other published relations but has the added credibility of being based on the most complete set of East Coast data. Bollinger's use of the actual intensity observations rather than the isoseismals permits the specification of fractile variations to the fit. Bollinger's 50 percent fractile relationship is

$$I = I_e + 2.87 - 0.0052 \Delta - 2.88 \log \Delta$$

I = site MM intensity

I_e = epicentral MM intensity

Δ = epicentral distance (km)

Figure 5.3 compares several of Bollinger's fractile relations with other recently published attenuation functions.

Figure 5.3
COMPARISON OF ATTENUATION FUNCTIONS

The acceleration relation that couples with Bollinger's relationship was derived from analysis on nearly 1500 world-wide accelerograms (USNRC, 1977). Extensive statistical analysis resulted in the following correlation

$$\log A_H = 0.14 I + 0.24 M - 0.68 \log \Delta + \gamma$$

A_H = peak horizontal acceleration

M = earthquake magnitude

γ = region-specific parameter.

Finally, we again use the Gutenberg-Richter relationship to relate earthquake magnitude to epicentral intensity,

$$M = 1.3 + 0.6 I_e$$

Combination of these three correlations results in

$$\log A_H = 0.47 M + 0.0905 - 1.08 \log \Delta - 0.0007 \Delta + \gamma.$$

The effect of the Δ term is small (it results in less than one percent g reduction at 500 km), and for convenience the term is neglected in the calculations.

The value of γ was determined by fitting this relationship to the only available acceleration data from Eastern/Central United States. (Herrmann et al., 1977 and USGS, 1976). Note from Table 5.3 that three instruments did not trigger during the March 25, 1976 earthquake. The accelerations at these stations are assumed to be just under the trigger level for the instruments, 1 percent g . Although the recording sites for these data are fortuitously very similar to the Babcock and Wilcox site, we allow for the possibility for some site response amplification in these data by considering three different fits to the data. We judge that the most appropriate fit to the data is for $\gamma = 0.9$, while consideration of possible site effects, particularly for data point 2, indicates that $\gamma = 0.8$ and $\gamma = 1.0$ are possible alternative fits. These three relationships are compared to the data in Figure 5.4.

Table 5.3
LOCATIONS OF ONLY APPROPRIATE
STRONG MOTION RECORDINGS

1	6 13 75	N MADRID, MISSOURI		
2	3 25 76	ARKABUTLA DAM, MISSISSIPPI	L	TOE
3	"	"	L	CREST
4	"	"	R	ABUT
5	"	TIPTONVILLE, TENNESSEE		
6	"	N MADRID, MISSOURI		
7	"	WAPPAPELLO DAM		CREST
8	"	"		TOE
9*	"	MEMPHIS, TENNESSEE		
10*	"	SARDIS DAM, MISSISSIPPI		
11*	"	POPLAR BLUFF, MISSOURI		

*Instrument, set at 1 percent g, did not trigger

Figure 5.4
COMPARISON BETWEEN
ATTENUATION FUNCTION AND DATA

As Figure 5.4 suggests, it is very important to consider the magnitude of the data dispersion about our mean attenuation relationship. Each of the component relationships that were synthesized into our attenuation relationship were themselves best fits to data with associated dispersion. Because the data sets used in these individual analyses is diverse and not readily available, we choose to assess the dispersion for our attenuation relationship through consideration of other data sets and other attenuation analyses. The statistical properties of peak acceleration are usually characterized in terms of the natural logarithm of acceleration and thus the dispersions are dispersions of $\ln(A_p)$. Typical standard deviations of this parameter range from .51 (McGuire, 1974) to 1.2 (Esteve, 1970) with a median value close to the value of 0.707 determined by Donovan (1974). We judge, therefore, that the value of 0.707 is a reasonable best estimate of the one standard deviation dispersion on acceleration.

Because the data base from which our attenuation relation was derived consists of predominantly far-field accelerations, the relation is less valid in the near-field. We account for this by limiting the peak accelerations in the near field. Because of the aseismicity of the host region and the distance from the site to significant sources, the details of this near field response are inconsequential. This complete attenuation relationship is presented in Figure 5.5 for several magnitudes.

RESULTS

The results were obtained by computer calculations with a risk analysis code (McGuire, 1976b) that is based on the work of Cornell (1968). The basis for this approach was summarized in Section 2.0.

As described in Section 2.0, the computer code calculates, for circular sectors within each source region at the site, the expected number of earthquakes causing accelerations greater than a specified acceleration and this is done for each source region and the host region. The expected numbers are summed for each region, and the resulting risk calculated from

$$\text{risk} = 1.0 - \exp(-\text{total expected number}).$$

Figure 5.5
ATTENUATION RELATIONSHIP USED
IN THE ANALYSIS

The return period associated with the specified acceleration is then the reciprocal of the risk. It follows from the definition of return period that accelerations with a particular return period have a 63 percent probability of being exceeded within the return period.

Our estimate of the seismic risk represents the weighted results from five individual calculations. The five calculations represent a base case and four perturbations of input parameters about this base. The perturbations are weighted by subjective estimates of their probability of occurrence and thus their weighted combination can be considered a Bayesian estimate.

The parameters that are considered uncertain and which are included in the Bayesian estimate of the risk are the maximum earthquake in each source zone and the intercept of the attenuation relation expressed through the value of γ .

The base case is considered to consist of the following input:

- maximum earthquake = largest historical plus one-half magnitude unit
- attenuation intercept, $\gamma = 0.90$

We characterize the uncertainty in these data by considering that it is roughly 70 percent probable that the maximum earthquake is as specified above, and that it is 15 percent probable that the maximum earthquake is respectively one-half magnitude unit larger and one-half magnitude unit smaller.

Similarly, we consider the value $\gamma = 0.9$ to be also 70 percent probable with perturbations of $\gamma = 1.0$ and $\gamma = 0.80$ to be respectively 15 percent probable. The best estimate in Figure 5.6 is the weighted summation of these five calculations. The plus one standard deviation is derived from 50 percent-50 percent weighting of the two more conservative runs and similarly for the minus one standard deviation.

Figure 5.6

RETURN PERIOD - PEAK ACCELERATIONS
FOR THE WESTINGHOUSE FACILITY

TERA CORPORATION

We have further examined the sensitivity of the results to possible uncertainty in other elements of the input and have established that the return period accelerations are relatively insensitive to reasonable variations in

- The slope of the attenuation function;
- The boundaries of the source zones; and
- The historical earthquake statistical parameters within each zone.

For example, when the slope of the attenuation function was varied 25 percent to reflect the use of the simpler and less refined relation (USNRC, 1977)

$$\log A_H = 0.25 + 0.25 I$$

in place of the $\log A_H$ relation above, the peak accelerations at a given return period increased only 10 percent. The insensitivity of the results to source zone geometry and earthquake recurrence intervals can be established through a parametric survey, but this can also be concluded from a careful examination of the results of a single run. It turns out that generally each source region (including the host region) contribute equally to the risk at the site up to the 500-year return period earthquakes. The risk above the 500-year return period is increasingly dominated by the New Madrid Seismic Zone but for shorter return periods, the contributions to the risk are about the same for nearby small earthquakes and distant, large earthquakes. Since no single zone dominates, small changes in the geometry and earthquake statistics for any single zone do not significantly change the risk.

COMPARISON WITH OTHER DESIGN CRITERIA

A better perspective of the methodology and these results can be obtained through comparison with other seismic design standards.

A commonly cited design standard is based on the analysis of Algermissen and Perkins (1976). This analysis, which was probabilistic and used the same seismic zonation as our analysis, determined that the site could experience up to 4 percent g with a return period of 475 years. This is roughly one-half the 475-year acceleration that our analysis predicts. The difference results principally from the manner in which dispersion is included in the analysis. Our analysis allows for one standard deviation acceleration of a factor of two over the mean whereas Algermissen and Perkins neglect this dispersion. When we neglect dispersion in our model, we calculate a 3 percent g peak acceleration, a result quite consistent with the Algermissen and Perkins result.

Finally, it is interesting to compare our analysis against the standards for nuclear power plants. The design methodology for nuclear power plants is deterministic, requiring identification of the largest credible earthquake which is assumed to occur either at the nearest provincial boundary, or at the site. As such, this methodology is incapable of expressing the criteria in terms of a return period acceleration. It is, therefore, virtually impossible to quantitatively compare results for specific sites. Typical values of the SSE acceleration for Eastern United States plants range from 10-20 percent g and our results indicate that this corresponds to return periods from 1,000 to 10,000 years.

RESPONSE SPECTRUM

These results define the peak horizontal acceleration at the facility for various return periods. We have also determined an appropriate response spectrum for the site since some structures and equipment at the Westinghouse facility have sufficiently low fundamental frequencies to experience spectral amplification of the ground motion. The response spectrum for the site clearly cannot be developed in association with a specific earthquake; our return period accelerations represent an integrated effect at the site from an extraordinarily variety of earthquakes and the response spectrum must reflect this. Accordingly, we judge that the shape of the spectrum should be similar to the Newmark-Blume

statistically-based spectra from which Regulatory Guide 1.60 evolved. This intuitive judgment is supported by a separate analysis we performed in which the risk is assessed through peak velocity. This calculation, which used McGuire's velocity attenuation relation (1976a), shows that the ratio of peak velocity to peak acceleration is very close to the ratio developed in WASH 1255 for alluvial sites. Because of an almost total lack of good data, the absolute level of spectral accelerations appropriate for design is very difficult to determine. For example, it is well known that attenuation in the Eastern United States is much less rapid than in the West (Alsup, 1972). Since the basis for Regulatory Guide 1.60 is exclusively western data, one might argue that the appropriate response spectral amplitudes should be in excess of the mean, perhaps the one standard deviation level, to account for the lesser attenuation. Alternatively, given the objective of best estimate results with minimum conservatism, it could be argued that the mean response spectrum for alluvium in WASH 1255 is most appropriate. There is, unfortunately, very little quantitative basis for choosing between these alternatives.

In our final consideration, we emphasize two points. First, the controversy surrounding the nature of attenuation (Q and its possible frequency dependency) and second, recent calculations at Lawrence Livermore Laboratory which show that the effects of straight line approximations of statistical spectra make Regulatory Guide 1.60 slightly more conservative than a one standard deviation spectrum. Accordingly, it is our judgment that the mean response spectrum for alluvium presented in WASH 1255 is the most appropriate for analysis of the Westinghouse facility. The uncertainties in this judgment are, however, large; there is probably a 20 percent probability that the spectral accelerations could exceed the 75 percentile accelerations in WASH 1255.

In summary, we have combined the best available input data with the most credible tools of seismic risk analysis to determine the return period of acceleration at the Westinghouse facility. The results, shown in Figure 5.6, account

for the dispersion of the data about the functional relationships used in the model. Further, the results are insensitive to variations in the source zone geometries or seismic histories. *Response spectral accelerations* can be determined by scaling the mean response spectrum in WASH 1255 to the desired peak acceleration.

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