BEFORE THE STATE OF WASHINGTON
ENERGY FACILITY SITE EVALUATION COUNCIL

IN RE APPLICATION NO. 99-1 SUMAS ENERGY 2 GENERATION FACILITY	EXHIBIT _________ (AP-T)

APPLICANT'S PREFILED TESTIMONY

ALLAN PORUSH

Introduction

Q. Please state your name and business address.

A. My name is Allan Porush. My business address is URS Corporation, 911 Wilshire Boulevard, Suite 800, Los Angeles, CA 90017.

Q. What subjects do you intend to address in your testimony?

A. First, I will address my background as a structural engineer specializing in the design of structures to resist earthquake motions, particularly industrial facilities. Also, I will

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discuss my background in developing the seismic provisions contained in current building codes (such as the Uniform Building Code).

Second, I will describe the process by which data is normally obtained for use in structural design to resist seismic ground motions.

Third, I will discuss the differences between the Uniform Building Code's Zone 3 and Zone 4 classifications and my involvement in the development of the UBC's seismic provisions which invoke these classifications.

Fourth, I will address statements made by Professor Donald J. Easterbrook regarding seismic risks to the SE2 project, as well as the principal types of seismic risk to structures and equipment such as are proposed for the SE2 plant and how such risks are generally addressed through structural engineering.

Background

Q. What is your position at URS Corporation?

A. I am a Principal Structural Engineer with URS. I serve as a consultant and/or as a Project Manager depending on the nature of the project.

Q. Could you describe your background and experience?

A. My education includes two degrees in Civil Engineering from the California Institute of Technology (Caltech) and an additional degree (a Professional Engineer's Degree) in Structural Engineering from the University of Southern California (USC).

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I have been working as a Structural Engineer for over 40 years. For the first 25 years, I was a Structural Engineer with a large Los Angeles-area engineering/ construction company called C. F. Braun & Company. Braun specialized in the design and construction of large industrial facilities, such as petro-chemical plants, oil refineries and power plants. At Braun, I specialized in issues such as design of structures to resist forces from wind, tornado, and earthquake.

In 1986, I joined URS as a Principal Structural Engineer in the "Earthquake Engineering Group." For the past 15 years, I have been a senior consultant in this Group, dealing primarily with issues involved in the design of buildings and industrial facilities to resist earthquakes. I have been heavily involved in reviewing and assessing the risks of seismically induced damage to a large number of facilities ranging from hospitals, aerospace companies, refineries and power plants. Very recently, I have been involved in the preliminary/conceptual structural/seismic design of a power plant in Mexico, and in the structural/seismic review of equipment at two newly designed natural gas "peaking" power plants in Southern California.

Another key aspect in my background is the role that I played from 1984 through 1996 in the organizations that develop and promulgate the seismic provisions contained in building codes. In particular, I was first a member and then chairman of the Seismology Committee of the Structural Engineers of California (SEAOC). During that time, the SEAOC Seismology Committee developed the seismic provisions of the 1988 Uniform Building Code (UBC), and during my chairmanship,

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spearheaded their adoption by the International Conference of Building Officials (ICBO) into the 1988 UBC. Subsequently, I served as the SEAOC representative to the Building Seismic Safety Council (BSSC), the national council funded by FEMA and charged with the development of nationally applicable building code provisions. I served as Vice-Chairman of the Board of Directors of the BSSC from 1992 until 1996.

A copy of my curriculum vitae is attached as Exhibit AP-1.

Seismic Structural Design

Q. How do you determine whether a facility can be designed to withstand seismic hazards?

A Let's assume that we are talking about a major facility, and not just a small strip mall. For such a major facility, a structural engineer first needs to know whether and to what extent the various seismic hazards will occur. Such seismic hazards include the likelihood and the extent of soil instabilities (including liquefaction and dynamic settlements), as well as the probable severity of ground shaking. Usually, the seismic hazard which dominates the structural design is the level (or intensity) of ground shaking to be designed for. The Uniform Building Code provides a first level of definition for the design motions and forces, which is adequate for most buildings. For some facilities, like power plants, however, a structural engineer will usually require a more site-specific characterization of the probable level of ground shaking that could affect the site. Such a characterization of ground shaking is commonly

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expressed in probabilistic terms (e.g., a ground motion intensity having a 10 percent probability of being exceeded in a 50-year period.).

This information is generated through three different interacting disciplines. A Geologist locates and characterizes nearby faults and a Geotechnical Engineer makes a preliminary determination of the type of soil at the site (whether the material underlying the site is sand or clay, and how dense or firm (or alternately, how loose) the material is). The Geologist and the Geotechnical Engineer provide this information to an Engineering Seismologist (sometimes called Earthquake Engineer). The Engineering Seismologist then characterizes the likely ground motion intensity. After the probable intensity of ground motion has been characterized, the Geotechnical Engineer revisits the soil profile to assess whether liquefaction (or dynamic settlement) is likely to occur. If it is likely, then he determines which soil layers are susceptible, and estimates the extent of the liquefaction likely to occur in those layers for the estimated magnitudes of potential earthquakes and the intensity of ground motion that each might produce.

Depending on the depth and extent of the liquefaction, various engineering approaches may be taken so that a power facility can resist the effects of such liquefaction. These will range from merely designing extra flexibility into connecting piping, to putting the major equipment on deep foundations such as steel piles. Armed with the above information, the Structural Engineer can determine whether a facility can be designed to withstand seismic hazards at the proposed site and proceed with structural/seismic design of the facility.

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UBC Seismic Provisions

Q. You mentioned that in 1988, you worked on the Committee that developed the seismic provisions of the 1988 Uniform Building Code. Please describe that work.

A. The Seismology Committee of the Structural Engineers of California (SEAOC) developed the seismic provisions of the 1988 Uniform Building Code. Local SEAOC Seismology Committees developed these provisions and then submitted to the SEAOC State Committee for debate and approval. When approved by the SEAOC state organization, SEAOC submitted these seismic provisions to the International Conference of Building Officials (ICBO), the publisher of the UBC. The SEAOC-submitted seismic provisions were then adopted into the Uniform Building Code at an ICBO national convention in early 1989.

I was a member of the local Los Angeles Seismology Committee starting in about 1979, and a member of the State Committee from 1982 through 1988. Finally, I served as Chairman of the SEAOC Seismology Committee in 1988 — the year the final provisions were submitted to ICBO for inclusion in the UBC. I had to defend the SEAOC submittals in the ICBO Committees, and then again at the ICBO national convention.

Q. What is the SE2 site's classification for seismic conditions under the UBC?

A. Zone 3.

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Q. Can you explain what that means?

A. The UBC Zones are based on an assessment of the likely peak ground acceleration in an area. The peak ground acceleration is determined based on a certain standard probability that it will be exceeded in a 50-year period. I mentioned above that this standard probability is expressed as follows. We select a level of peak ground acceleration having a 10 percent probability of being exceeded in a 50-year period. The basic map which provided contours of the peak ground accelerations at this specified standard level of probability was initially developed by the United States Geological Survey.

Q. How would a change in classification from Zone 3 to Zone 4 affect the design of a facility like SE2?

A. Zone 4 is defined as those areas where the probable peak ground acceleration exceeds 0.3g (where "g" represents the acceleration of gravity). Zone 3 is defined as those areas not assigned to Zone 4 where the probable peak ground acceleration exceeds 0.2g.

A change in classification of the Sumas area from Zone 3 to Zone 4 would raise the design force level by 33 percent. In Zone 3 the design force is based on a design peak ground acceleration of 0.3g. In Zone 4, the value for design is 0.4g.

If a site-specific assessment by a competent earth-science team is to be made, then I believe that such an assessment would be far more meaningful than the "broad-brush"

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assessment made by the USGS and then “smoothed” by the SEAOC Seismology Committee. It is my understanding that the plan for the SE2 facility is for a site-specific seismic hazard assessment to be made. Design will then be based on at least Zone 3 peak ground accelerations, or higher accelerations if they result from a probabilistic site-specific study. My feeling is that this provides you with the best of both worlds — a site-specific basis and a commonly accepted general standard to use as a criteria “floor” for the design of the facility.

Seismic Risks

Q. Can you address seismic hazards through structural design of a facility like SE2?

A. Yes, designing large industrial buildings like SE2 to withstand earthquakes is what I do every day, and it is almost always possible to address such hazards through structural design.

Q. What are the principal categories of seismic hazards that you must address in your design work?

A. There are basically six main categories of seismic hazards that can arise, although in most instances, for a site where such hazards are significant enough to warrant special building designs, only two or three will need to be addressed.

1. Ground Motion. In any earthquake, ground motion or shaking occurs, and as a general rule, this is a hazard that I will be designing for in virtually every project I work on.

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2. Landslide. If the site is located on or near a slope and depending on the soil conditions, landslides may be an earthquake hazard.

3. Liquefaction. If the site contains loose sandy soils below the water table, there may sometimes be the possibility of "liquefaction" occurring at different ground depths, times and locations during a sufficiently long and intense earthquake. For a general description of what liquefaction is, and how and when it may occur, I would refer you to my colleague, Mark Molinari, who I understand will be discussing this topic in his testimony.

4. Ground Rupture. Ground rupture refers to an uplifting or shifting of the earth along a fault line during an earthquake. Such earth movements can vary from a fraction of an inch to, in extreme cases, several feet. However, once again, I would refer to Mr. Molinari who I understand will also be addressing this issue in his testimony.

5. Lateral Spreading. Lateral spreading of a soil deposit can be caused by liquefaction, or it sometimes occurs where certain types of clay are present which can become soft and move under the action of the earth's shaking.

6. Dynamic Settlements. Finally, dynamic settlement refers a compacting of certain types of soils during an earthquake.

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Q. Are you familiar with Professor Donald Easterbrook's recent remarks regarding the seismic conditions near Sumas, Washington?

A. Yes, generally.

Q Professor Easterbrook has stated in reference to the SE2 project that "designing a large, earthquake-proof structure that could withstand ground failure is impossible" and that "<u>no</u> large structure can be designed to withstand sudden ground failure beneath its foundation." (Emphasis in original.) Do you agree with those statements?

A No.

Q Why not?

A The term "ground failure" as Professor Easterbrook has used it is excessively vague and undefined. There are wide ranges or degrees of various types of "ground failure," ranging from very minor to severe. It is neither rational nor meaningful to merely mention a type of soil behavior and then presume that this named behavior will always occur at the upper extreme of severity for such phenomena. Unless the soil behavior is at the extreme upper limit of severity, potential "ground failure" can be and is very commonly addressed through design/mitigation measures. Such measures have been successfully employed in seismic hazard environments significantly more severe than those that appear to be present in the Sumas Valley.

As to the vagueness and lack of definition, there are several types of ground failure, and it is not clear from Professor Easterbrook's statement what type he is referring to.

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In earthquake engineering, the more common types of ground failure include liquefaction and landslide, which Professor Easterbrook appears to treat separately. So apparently he means something else by "ground failure." I can only speculate that he may be speaking of phenomena such as dynamic settlements or lateral spreading.

In theory, dynamic settlement could be an issue, but it does not appear that the material under the site is sufficiently loose or unconsolidated so as to lend itself to such behavior. In any case, dynamic settlements are usually of limited magnitude. I am not aware of any cases of seismically induced dynamic settlements (that were not caused by liquefaction or landslide or direct fault offset) that exceed a few inches. It is therefore virtually always possible to address any possible dynamic settlement risks through structural design.

Regarding "ground failure," Professor Easterbrook refers to failures in the 1964 Alaskan earthquake where "the ground slid out from under the foundations." Most of the ground failures in Alaska emanated from either liquefaction or from a flow of a bluff or palisade (a vertical offset) consisting largely of soft, weak clays. It is my understanding that the SE2 site contains neither the vertical offsets of a bluff geometry nor the clays of the type that would be susceptible to such flows. Liquefaction may be an issue, but again, Professor Easterbrook seems to separate this issue from that of "sudden ground failure". I will have some additional comments on liquefaction below.

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The second problem that I have with Dr. Easterbrook's statement is that he appears to presume that if a phenomenon occurs, it will occur at the extreme upper end of severity for that type of phenomenon. This would be like saying that if it rains, the runoff will automatically be equivalent to the million-year flood.

Let's use liquefaction as an example. Even if a modest amount of liquefaction occurred at a site, the extent of liquefaction would vary in time as well as in space. Certain layers of soil might *partially* liquefy, and then other layers might require additional shaking before they would *begin* to liquefy. Thus, there is a very wide range of extent and severity between no liquefaction occurring and the entire site liquefying. The scenario of an entire soil profile at a site liquefying is extremely rare. In order for an entire soil profile at a site to liquefy, it would have to be characterized by very loose saturated granular material and very strong shaking of long duration. In my opinion, it is doubtful that all these conditions are satisfied at the SE2 site.

The next problem that I have with Professor Easterbrook's statement is that his general conclusion about not being able to design against soil failure is exaggerated to the point of being basically untrue.   His testimony contains statements such as "building a large industrial complex which stores hazardous materials on or in close proximity to active faults is not something one can engineer around," and "designing a large earthquake-proof structure that could withstand ground failure is impossible." I consider these statements to be untrue in all but the most exceptional circumstances, and although further geotechnical investigations will be conducted, it appears very doubtful to me that these exceptional circumstances are present at the SE2 plant site.

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Q. You mentioned that design measures have successfully addressed seismic risks in other places. Can you provide some examples?

A. Yes. Let me give three examples that I have recently been involved in.

First, above I alluded to my participation in the conceptual design of a new power facility in Mexico. This proposed facility (which may still be built) is, in fact, sited directly over a small active fault. Our conceptual design had to provide a foundation that would resist failure due to fault rupture directly under the foundation for the power train. This is expensive, and I don't normally recommend such an approach. But the point is, depending on the circumstances and the severity of the hazard, a statement that says an engineered solution is "impossible" is flat wrong.

A second recent example is one where I was asked to review the seismic design of a new power plant that was being located less than 1 kilometer from the San Andreas Fault near North Palm Springs, California. The estimated ground motion was extremely high — well over 0.5g peak ground acceleration. Suffice it to say, the plant design was performed so as to provide structural systems capable of resisting this very intense level of shaking.

A third example is one where we were asked to consult on the design of the Puerto Rico Convention Center in San Juan, Puerto Rico. This major structure was to be built at a site in which the top few feet could be expected to at least partially liquefy.

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We proposed and assisted in the design of a foundation scheme that used steel H-piles and that was capable of resisting the seismic forces.

The basic point here is that it may require special precautions, and it may be expensive to design to resist various seismic hazards, but depending on the extent and severity to which they occur, it is almost always possible to do so.

Q. What sorts of structural design features would typically be used to address potential liquefaction for facilities such as SE2?

A. This answer depends heavily on the location and extent of liquefying material. I do not believe the upper soil layers in the Sumas Valley are particularly susceptible to liquifaction. If the upper soil layers in the Sumas Valley turned out to be far more susceptible to liquefaction than they appear to be, then it might be necessary to design measures to mitigate the potential effects of liquefaction. Otherwise, it might not. Speaking generally, however, there are several approaches to arriving at such design measures where there is a potential for liquefaction. The first common approach is to remove several feet of the looser soil material. A second approach can be used if there is a distinct layer of material that is susceptible to liquefaction. This approach is to pressure grout this very susceptible soil layer. A third approach may be practical if only the top say 10 to 15 feet are particularly susceptible to liquefaction. This third approach is to locally de-water the site down to a depth where liquefaction is unlikely. A fourth approach might be to relieve the build-up of pore water pressure by installing geotechnical "relief valves" in the form of stone columns.

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Ultimately, the approach is usually to place the foundations for the main structures and equipment on piles. Such piles might, in a worst case scenario, need to extend through the softer material, perhaps as much as 50 or 60 feet down below grade into the more dense material. *See* Exhibit AP-2 attached hereto. In such a scenario, the piles would have to transfer both vertical and lateral forces to the soil profile. Although expensive, such an approach is feasible, and has been used on many projects in areas of much higher earthquake potential.

Q. From the materials you've seen regarding SE2, including the materials from Professor Easterbrook, is there anything that suggests to you it might be "impossible" to address liquefaction issues using structural design procedures at this site?

A. No. Addressing liquefaction is usually possible for most sites and most structures. This is particularly the case if the intensity of ground motion is relatively low, and/or the duration of strong shaking is low (such as from an event of Magnitude less than say 6.0). At worst, it might be necessary to place a power facility on pile foundations or to institute other mitigation measures.

I mentioned a couple of sites (such as the Puerto Rico Convention Center) where essentially the structure was supported on a series of steel H-piles that transferred both vertical forces (gravity and seismic) as well as lateral forces (seismic) into the soil profile. The seismic risks were addressed, and the resulting structure is not in any greater risk of damage than if there were no liquefaction potential. I note that the

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level of shaking hazard in San Juan, Puerto Rico is significantly higher than in northern Washington.

Q. Is it "impossible" to address surface rupture risks through building design?

A. First of all, I understand from Mr. Molinari's testimony that the possibility of fault rupture is not an issue at the SE2 site. That said, even if there were a possibility of such movement, it is an exaggeration to say that it would necessarily be "impossible" to address it through structural design. For example, I mentioned a recent project (which incidentally was a power generation facility) where we did design to directly resist the effects of fault surface rupture. Whether such a design would work for the SE2 site would depend on the nature and extent of any possible fault rupture.

Q. Are landslides a seismic risk that can be addressed through structural design, or is it "impossible" to design a building to cope with such risks?

A. It is my understanding that landslide (as the term is normally understood) is not an issue at the SE2 site. I understand that the site is relatively flat, and that there are no steep slopes directly adjoining the site. Even if the site were not flat, however, the answer would be no. The word "impossible" may apply if there are major mountains or hills with steep, unstable layers of soil bedding directly adjacent to the site. However, a statement that it is impossible to mitigate the risk is an exaggeration if the adjoining changes in elevation are on the order of say less than 100 feet, if the slope or soil bedding is adequately stable, and/or if the ground shaking is not too intense. If there are modest elevation differences to contend with, then benching, retaining walls, tiebacks, rock bolts, and other engineering solutions can usually mitigate the risk.

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Q. **Regarding shaking, Dr. Easterbrook has stated in his Affidavit that "shaking causes buildings to collapse because the intensity of the shaking exerts forces on the structural members of the building." Do you agree with this statement?**

A. Categorically no.

Q. Why is that?

A. The statement is a sensationalized, gross exaggeration of what happens in earthquakes. First let me state that I have investigated earthquake damage to structures in seven earthquakes, that I have performed a detailed forensic study of a collapsed structure in one of them, and we have had to repair structural damage in three such earthquakes.

As a generalization which is almost always true, well-designed and well-constructed structures do not collapse in earthquakes. Poorly designed or poorly constructed buildings are the ones that you may see in the newspapers.

I mentioned the forensic study that I performed. It was on a parking structure at California State University at Northridge (CSUN) which suffered a 50 percent collapse in the 1994 Northridge Earthquake. Note that this was very near the epicenter of the earthquake, and the ground motions were very severe. Our report concluded that the design of the CSUN parking structure clearly did not meet the "intent" of the building code, and furthermore, it was our opinion that it did not "meet" the requirements of the code. There was another parking structure two blocks

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away which experienced only minor damage. You had to look closely to see the cracks.   The key point is that a well-designed structure suffered only very minor damage, while it was only the clearly deficient structure which suffered partial collapse.

Structures that are competently designed for a level of ground motions commensurate with what they experience in an actual earthquake have been observed to perform well. We expect some distributed structural damage in strong earthquakes. However, to suggest that just because there is seismically-induced shaking, and “the shaking exerts forces on the structural members of the building,” that this leads to the scenario that everything collapses, is so grossly exaggerated as to be irresponsible.

END OF TESTIMONY

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