BEFORE THE STATE OF WASHINGTON
ENERGY FACILITY SITE EVALUATION COUNCIL

In the Matter of Application No. 99-1:

SUMAS ENERGY 2 GENERATION FACILITY

EXHIBIT ___ (AP-RT)

APPLICANT'S PRE-FILED REBUTTAL TESTIMONY

WITNESS: ALLAN PORUSH

Q. Would you please reintroduce yourself to the Council.

A. My name is Allan Porush. I am a Principal Structural Engineer with URS Corporation.

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

A. I will be commenting on the responses to my earlier testimony ("Porush PFT") made by Professor Don Easterbrook in his written testimony filed by Whatcom County on October 1, 2001 ("Easterbrook PFT"). I will also make a couple of comments

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regarding the written testimony of Professor David Engebretson, also filed by Whatcom County on October 1, 2001 ("Engebretston PFT").

General Comments

Q. What is your understanding of Professor Easterbrook's opinion as to whether it is possible to address the seismic hazards that may be present at the SE2 site through building design?

A. It appears that he believes it is simply impossible to design a building to address the seismic risks at the SE2 site.

Q. Do you agree with his assessment?

A. Absolutely not. Professor Easterbrook seems to have imagined the most extreme earthquake with the most extreme consequences and then made the assumption that there is no way to engineer against such extreme circumstances. This latter assumption may be partially true, but it isn't particularly relevant. I have instead tried to consider the more reasonably probable range of seismic effects and testified about the ability of structural engineers to guard against the consequences of these more probable effects with engineering measures.

I am not a geologist or seismologist, but I work with such experts on a daily basis, and I have a general understanding of the basics in each of these disciplines and what makes sense in terms of seismic hazards. I have reviewed the testimony of Mark Molinari, the reports and testimony of Professors Easterbrook and Engebretson, the sections of SE2's Second Revised Application regarding seismic issues and, most

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recently, the sections dealing with seismic issues in the Draft Supplemental Environmental Impact Statement ("D-SEIS") prepared by the Council's independent consultant, Jones & Stokes. Based on my review of these materials, I have the strong impression that the faults in the Sumas Valley are not even in the same league, either in overall potential or in probability of having an energy release, as those we very commonly design for in most of coastal southern California, for example, the San Andreas, Newport-Inglewood, Whittier-Elsinore, Hollywood-Raymond, and Rose Canyon faults. Based on my review of the information and analysis described above, I feel quite confident that the seismic risks that may reasonably be assumed to be present at the SE2 site can be addressed through engineering design.

Q. Are there any other aspects of Professor Easterbrook's testimony generally that you would like to comment on?

A. Yes. I was struck again by his tendency to presume that if it is remotely possible for a phenomenon to occur, we must presume that it *will* occur. In addition, Professor Easterbrook's approach seems to be to assume that if a phenomenon is in fact possible, it will automatically occur at the extreme upper end of severity for that type of phenomenon. Indeed, some of the extreme seismic scenarios Professor Easterbrook hypothesizes haven't occurred in the Sumas Valley in many thousands of years.

Q. Can you give any examples of this approach?

A. Yes. For example, Mr. Molinari testified, and Professor Easterbrook does not deny, that there is no indication of fault movement on the Sumas fault in the last 11,000

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years (within Holocene times). In California, a lack of movement within the Holocene is an adequate basis for designating a fault as "inactive." However, despite the lack of fault movement, not only is Professor Easterbrook suggesting that surface fault rupture is "possible," he is projecting fault ruptures towards the upper end of the scale for this type of phenomenon, namely, "fault rupture of 15 to 20" feet. Easterbrook PFT, p. 17:3.

Another "possible" hazard that Professor Easterbrook carries to an extreme is the extent, the severity and the consequences of liquefaction. We have agreed that there may be a liquefaction potential at the SE2 site, and we are the first to insist that it must be addressed in the design of a power generating facility (or any facility for that matter). However, Professor Easterbrook seems to suggest that the entire site, apparently down to the full 1,000 feet of alluvium, will turn to soup, and that this precludes the site from being a candidate for siting a power facility. In the real world, the extent and depth of liquefaction are limited, may occur in some soil layers and not others, and can almost always either be mitigated or designed for.

<u>Landslides and Fault Rupture</u>

Q. Would you please remind the Council which seismic risks Professor Easterbrook says the Council should be concerned about with respect to the SE2 site.

A. Professor Easterbrook says that there are four risks: landslides, fault rupture, ground shaking, and "ground failure," *i.e.*, apparently, liquefaction.

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Q. With regard to the first of these risks, in your earlier testimony, you said that it was your understanding that due to the topography of the SE2 site and surrounding area, landslides are not an issue. In responding to your testimony, Professor Easterbrook says that this incorrect because landslides several miles long have occurred in other parts of Whatcom County and therefore "must be considered possible" at the SE2 site. Would you care to comment on this response?

A. Yes. Mr. Molinari will address this point in his testimony, but I did notice that the Council's independent consultant agrees with Mr. Molinari that since the "project site is in a broad flat-lying valley" with no steep adjacent grades, "seismically induced slope failures are not a consideration at the site . . . ." D-SEIS, p. 3.7-9. In fact, although he continues to insist that landslides pose a risk, Professor Easterbrook himself states that he and his colleagues "do not believe that the seismically-induced landslide potential is prohibitively high at Sumas . . . ." Easterbrook PFT, p. 9:8. Based on all these statements, I continue to believe that the risk of landslides should not be an issue which enters into a decision as to whether a power facility should be located at the SE2 site.

Q. Professor Easterbrook criticizes your testimony regarding the risk of fault rupture because "Mr. Molinari's testimony is inaccurate on this point and fault ruptures of 15-20 [feet] must be considered possible." Would you like to comment on this response?

A. Yes. I am relying on Mr. Molinari's testimony both because I am familiar with and have the utmost confidence in his skills as a geologist and because his conclusions

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seem far more reasonable to me than Professor Easterbrook's. Mr. Molinari concludes that the likelihood of surface fault rupture affecting the site is very low, particularly considering the lack of evidence that the Sumas fault is active. Also, I again noticed that Jones & Stokes agrees with Mr. Molinari that "[t]he potential for damage to the plant site or pipeline by fault rupture is considered highly unlikely because of the lack of any evidence of geologically recent surface faulting in the project vicinity." D-SEIS, p. 3.7-8.

In contrast, Professor Easterbrook begins with the questionable conclusion that the Sumas fault is active; then based on this conclusion, assumes that the fault will rupture at the surface; next theorizes that if this relatively short, little known fault ruptures, it will not be a minor or even moderate rupture of a few inches to a few feet but, rather, a very large offset of 15 to 20 feet; and finally, wraps up this astonishing chain of conjecture by insisting that the hypothesized event will occur immediately underneath the SE2 facility. Once again, from my standpoint as a structural engineer who for the past 40 years has specialized in the seismic design of large industrial facilities, often working on a daily basis with professionals in a range of related specialities, including geology, I cannot consider Professor Easterbrook's professed concern about the possibility of an enormous fault rupture directly beneath the SE2 facility as anything less than incredible.

<u>Seismic Shaking</u>

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Q. What about the other two risks that Professor Easterbrook points to, ground shaking and liquefaction — are these risks that need to be considered with respect to the design possibilities for the SE2 facility?

A. I understand that there is a reasonable probability that the site will, at some time in the life of the proposed facility, be subjected to seismically induced ground shaking. It is also my understanding that the soil profile contains some loosely consolidated granular material below the water table. This latter circumstance suggests that the phenomenon know as liquefaction might occur in a strong earthquake of long duration.

Q. As to the first of these two risks, do you believe that the ground shaking hazard at the SE2 site is unusually high or severe?

A. No. Once again, I am relying on the assessment of the geologists and seismologists, including those in the U.S.G.S., who have characterized the potential for strong seismic ground motions in the region as a whole. Considering seismic ground motions I do not believe that this hazard is higher — in fact I do not believe it is as high — as we very commonly design for in most of coastal southern California.

Q. Dr. Easterbrook has suggested that "if the shaking forces exceed the strength of the structural members, they will fail . . . a well known fact of basic physics." Do you agree?

A. First of all, I take issue with this alleged "fact of physics," which is expressed in a grossly oversimplified fashion. Structural/seismic design requires that the structural engineer supply not only strength but also *ductility* to his structure in order that it is

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able to successfully resist earthquakes. Therefore, one of the most important aspects of design for many structures is assuring that elements may reach their limits of strength, and then yield and absorb energy without failure. Designing structures to resist the forces (and/or the energy) generated by earthquake-induced ground motions is what structural/seismic design is all about. It's what we as structural engineers do, and for the most part, do very well. I believe that if you had driven around the epicentral region of the Northridge earthquake, for example, you would have been struck not by how extensive the major damage was, but how limited it was.

Again, poorly designed structures may fail in strong earthquakes. Structures that are properly designed to resist earthquakes and then competently constructed, in an overwhelming percentage of cases, do not fail in earthquakes. They may suffer some damage, and they may require some repair, but they very seldom fail. In suggesting otherwise, I believe that Dr. Easterbrook is very much mistaken, and also very much outside his area of expertise when he comments on structural design and what is known, what can be done, and how reliable it might be.

Q. Does this mean that you can design "earthquake proof" structures as Professor Easterbrook suggests?

A. I have never used the phrase "earthquake proof." However, I believe that in the year 2001, the basic knowledge and the technology exist to adequately design structures such that, to a very high probability, they will not fail in an earthquake that is comparable in intensity to that of the "design" earthquake.

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Q. What do you mean by the "design" earthquake?

A. The design earthquake is the level of shaking intensity that the engineering seismologist determines to have the required probability for that site. That is, the building may be required to withstand, for example, the intensity of shaking produced by an earthquake that is expected to occur only roughly once every 500 years. This "design" earthquake is determined by conducting a site specific probablistic seismic hazard assessment ("PSHA"), which I understand SE2 has committed to performing prior to final design of the facility. *See* SE2's Second Revised Application, p. 2.15-3.

Q. Professor Easterbrook has indicated that "until a structure actually survives a certain ground acceleration, we have no way of knowing whether or not the design was successful." Do you agree with this statement?

A. No. We design and build in areas of high seismicity. We design buildings and industrial facilities including oil refineries and power plants. We design water systems, aqueducts and pipelines. The power plants include nuclear power plants. The industrial facilities include LNG and LPG facilities. We have the technology that allows us to design, to analyze and predict seismic response, and we now are beginning to have sufficient actual earthquake response data to correlate with our analyses. Professor Easterbrook's comment seems to suggest that nothing should be built because nothing is known about how structures respond to earthquakes and it is not possible to determine how they might respond. I consider this attitude to be totally misguided.

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Q. Are you saying that we have physical evidence which provides a basis for predicting how structures will behave in future earthquakes?

A. Today we have both analytical tools and a fair amount of actual earthquake response information to correlate with our analyses. We now can get a pretty good idea how a structure will behave in an earthquake and have a reasonable degree of confidence that it will behave roughly as we predict.

Q. Can you give an example of this sort of physical evidence?

A. Yes. I was on the EERI (Earthquake Engineering Research Institute) post-earthquake damage assessment team for industrial facilities after the Loma Prieta earthquake in 1989. In that capacity, our "squad" was assigned the task of reviewing Pacific Gas & Electric's Moss Landing power plant south of Monterey, California. In an environment of very strong shaking, the Moss Landing plant suffered minimal damage. One water tank ruptured, and we saw a few pipe supports that were bent. (I don't believe that the water tank was designed for earthquake.) Outside of those items, there was almost no discernible damage within the plant itself. The key conclusion is that even in an environment of very strong shaking, competent design leads to structures, including equipment and piping, which successfully resist the forces from an earthquake.

Q. Professor Easterbrook has noted that buildings in Loma Prieta, Kobe and Anchorage failed in past earthquakes. Doesn't this mean that buildings fail in earthquakes?

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A. As I indicated in my previous testimony, with very few exceptions, the structures that have failed in past earthquakes due to ground shaking were inadequately designed or poorly constructed. For example, I had a contract to perform a forensic review of the partial collapse of a parking structure on the campus of California State University at Northridge (CSUN) that failed during the 1994 Northridge earthquake. My paper on this structural failure was published in the Proceedings of the 1995 Annual Convention of the Structural Engineers of California. Our conclusion was that the structural/ seismic design of the failed CSUN parking structure did not meet the building code and was further deficient in several other respects. These design deficiencies were the direct cause of its failure. Another parking structure, which was located about one block away but which was correctly designed for seismic resistance, suffered only minor cracking. And this was in response to ground shaking at the CSUN campus with a peak ground acceleration that probably exceeded 0.5g.

There were perhaps 75 buildings on the CSUN campus. Two buildings (including the parking structure) partially collapsed, and about eight suffered unacceptable damage. I believe, based on today's standards, that the design was flawed for all of the buildings which suffered unacceptable levels of damage. I further believe that today, most if not all of those ten buildings could (and probably would) be designed such that in another Northridge earthquake, they would suffer only repairable damage.

The key point is that structural failure, even in a very extreme environment — directly at the epicenter of a Magnitude 6.5 event — is actually rare. And those failures that

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do occur can, with very few exceptions, be demonstrated to be because of inadequate design and/or construction.

Liquefaction

Q. The remaining seismic risk discussed by Professor Easterbrook is liquefaction. How deep do you usually expect liquefiable soil to exist?

A. In typical cases, you do not expect liquefaction to extend more than say 30 or 40 feet below ground surface.

Q. Professor Easterbrook in his testimony has emphasized the point that there is a 1,000 feet of alluvium in the Sumas Valley. Is that significant with respect to liquefaction?

A. No. Basically it is irrelevant. You typically do not get more than the depth that I have mentioned suffering liquefaction, even in very strong earthquakes. It doesn't matter whether there is 50 feet, 100 feet, or 1,000 feet of alluvium. The material at depths greater than the depth at which liquefaction occurs still retains its inter-granular forces, and therefore it retains its ability to carry load.

Q. From a structural engineering perspective, what can be done if there is the potential for liquefaction to affect a site?

A. It depends on the extent and the location of the material that appears at risk to liquefaction. As Mr. Molinari has testified and as I indicated previously, there are usually specific layers that are shown to be at risk — not an entire soil profile.

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Q. What do you do in typical cases?

A. First, somebody needs to assess the extent and the depth of potential liquefaction. That means that a competent geotechnical engineer must determine the soil properties, and then perform an analysis to assess this potential. If the liquefaction potential is judged to be moderate in its extent (as I said, it is my experience that usually only specific soil layers prove to be at risk to liquefaction), then the measures taken will vary with the extent, location, and depth of the potentially liquefying layer. I mentioned several such measures in my earlier prefiled testimony.

Q. What were some of them?

A. First, if the soil layer that is susceptible to liquefaction is only a few feet thick and near the surface, removal is sometimes considered. If only a specific layer is the culprit, then either grouting or installation of what I called "geotechnical relief valves" in the form of stone columns have been used. This latter technique allows the pore water pressure to dissipate before liquefaction can occur. I have seen dynamic compaction techniques used very effectively to densify the top say 10 to 15 feet of soil.

If the extent of liquefaction is too great for any of the above measures, the usual approach is to place structures on piles. The pile lengths required will vary with the soil profile, the depth to which liquefaction occurs, the severity of the ground shaking at the site, and the magnitude of the causative earthquake because of the time dependency. It is also possible that piles will have to be used in conjunction with some ground improvement method.

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Q. Professor Easterbrook has commented that piles will not be effective, because the fine-grained materials extend to such great depths. Do you agree with this?

A. No. First, the soils that are most prone to liquefaction are cohesionless granular soils, and not specifically fine-grained materials. Second, while I have not studied the SE2 soil profile in detail, as mentioned above, the zone of liquefaction is usually finite and below that zone, soils are usually still competent. In other words, the depth of loose granular materials does *not*, in and of itself, define the hazard. The other parameters, including the depth to the specific soil layer, the gradation and looseness of the material, the existence of any cohesive material, the intensity of ground shaking and the magnitude of the earthquake (*i.e.*, the duration) are all critical parameters in determining whether there is a risk that liquefaction may occur or not, and if it does occur, to what extent and to what depth.

The point is that normally, liquefaction only occurs down to a certain depth, and below that depth the soil retains its ability to carry load. Depending on the soil strength and depth, a typical situation might require that piles extend about 15 to 20 feet deeper than the deepest liquefiable layer.

Q. The piles do not have to extend to rock?

A. No, that is usually not necessary. In two cases that I recently had to deal with, friction piles that ranged from 40 to 60 feet in length were determined to be adequate. I do not know what length would be required at the SE2 site. It might well be that greater lengths are required at SE2. However, I have no doubt that at *some* length, piles

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would carry the loads into material that is not liquefied. In other words, it is almost always possible to design to resist liquefaction.

Q. Is there any doubt in your mind that a design solution to mitigate against the effects of liquefaction is almost always possible, and that such a solution will make structural failure very unlikely?

A. If the hazard is defined by seismic shaking together with a potential for portions of the soil profile to liquefy, and the geotechnical evaluation, the structural design and the construction are all done by competent practitioners, I have no doubt whatsoever. A design can be developed that will make failure very unlikely and damage, if not prevented, minimized and controlled.

Q. Is there any difference between designing for seismic motions with or without the occurrence of liquefaction?

A. Not in basic concept. The details of the design and of the analysis to verify the design may differ. Conceptually, however, a structural engineer will do the same thing in both situations. He will determine the forces on his structure, and then design first a structure and then a foundation system (with piles or otherwise) that will deliver the resulting forces to a soil medium capable of resisting those loads. If the potential liquefaction is extensive, then the foundation (such as piling) may be extensive and expensive. But in basic concept, a structural engineer does the exact same task whether or not liquefaction must be resisted.

Q. Is structural design for the effects of liquefaction commonly done?

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A. During the past ten years, it has become fairly common. Previously, although liquefaction was understood, it was much less common. In fact, requirements for design to resist the effects of liquefaction have only been in the building codes for the past eight years.

Q. Does that mean that structural failures in Alaska (in Anchorage), in the Marina district in Loma Prieta, and in Kobe that took place in areas where soil liquefaction occurred were of structures that predated design for liquefaction?

A. That is absolutely correct. That is one reason that pointing at structural failures say in the San Francisco Marina district and suggesting that these are “proof” that adequate structural designs cannot be developed are either naive or misguided.

It may be true, as suggested by Dr. Engebretson, that damage in areas where liquefaction has occurred are highly correlated with the severity of the liquefaction. However, I find that correlation irrelevant if the structures were not designed to resist liquefaction effects. For example, the damage to the Cypress freeway in Oakland was largely due to an exceptionally poor structural configuration. Yes, it may have been avoided if the liquefiable soil had not been present, and then again it might still have failed. It was a poor design both structurally and in not accounting for liquefaction effects.

Q. Professor Engebretson states in his testimony that accelerations are likely to be higher in soft soils than in areas underlain by bedrock, and that damage

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correlates with areas with thick unconsolidated sediments. Do you agree with these observations?

A. That is a very oversimplified view. His statements are probably true at low strain levels that are within the linear range of the foundation material, and in addition they are probably true for longer period structures. For higher strain amplitudes, where non-linear effects become important, and/or for structures having lower natural periods (higher natural frequencies), the reverse may be true. In other words, a stiff non-ductile structure may be in greater danger on rock than on a soft soil. Thus, large deposits of alluvium *might* cause the seismic motion to be amplified in certain period ranges and at low strain levels.

That said, if there is a possibility of higher accelerations, these should be correctly reflected in any properly calculated site-specific design basis. A properly developed design basis followed by a competent design means that there will be no increase in risk relative to any other site. Therefore, the higher acceleration (in whatever period range) and/or the propensity for greater damage is basically irrelevant in assessing whether the site should be considered or excluded from being considered as a candidate for a power facility.

Risks to Public Safety

Q. Professor Easterbrook has made an issue of the risk of danger to the public, and of "the irresponsibility of endangering the lives of innocent people" by placing a facility with "hazardous materials" on a site "where a geologic hazard could disrupt the service provided, or create and unacceptable risk of harm to the

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people in the town of Sumas." Do you have any comments regarding these alleged risks?

A. Yes. The sorts of seismic activity that seem reasonably probable at the proposed site present primarily business risks as opposed to wide spread risks to public safety. If the facility were not designed properly, it could be damaged during an earthquake, requiring the owner to suspend operations and perform expensive repairs. The same could be said of other commercial and industrial facilities in the area. The facility does not present unique public safety risks in the event of an earthquake. There are numerous natural gas pipelines in the Sumas area, both transmission lines and local distribution lines, as well as another natural-gas fired generating facility. The SE2 facility and pipeline, like these other pipelines, will presumably include seismic design features, as well as a variety of emergency shut-off protections. With the exception of natural gas, there are not significant quantities of hazardous materials stored at the site. There is an aqueous ammonia tank, but aqueous ammonia does not present a hazard to the general public and would further be surrounded by a containment area. Thus, I simply do not understand what Professor Easterbrook is talking about when when he refers to the risk of "endangering the lives of innocent people" or the danger of "hazardous materials" at the SE2 site.

Q. In addition, Professor Easterbrook has stated that the geologic hazards could "disrupt the service provided" (*i.e.*, power supply). How do you respond to this testimony?

A. I agree that some interruption of power generation is possible, as it is possible at any facility, although proper design will minimize the risk. The Moss Landing Power

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facility that I mentioned above was out of service for about three or four days after the Loma Prieta earthquake. However, this was not because of structural damage to the main generating facility, but rather because of damage to some electrical equipment, (transformers, etc.) that were part of a distribution sub-station outside of the basic plant boundaries.

It is my strong belief that the SE2 facility can be designed so as to not be subject to structural failure. It is also my belief that any damage that occurs should be controlable and, for most structural elements of the facility, repairable. Moreover, the degree of damage that might be incurred by a power facility in the Sumas Valley should be no greater than might be incurred by a facility at any other site in a seismically active zone that is subjected to strong seismically-induced ground shaking. In any event, the possibility of service interruption presents a business risk, not a safety concern.

END OF TESTIMONY

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