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16 February 2001

U.S. Army Corps of Engineers
Regulatory Branch
P.O. Box 3755
Seattle, WA 98124
ATTN: Jonathan Freedman, Project Manager

Washington State Department of Ecology
Shorelands and Environmental Assistance Program
3190 - 160th Ave. SE
Bellevue, WA 98008
ATTN: Ann Kenny, Environmental Specialist

Subject: Comments on Seattle Tacoma International Airport Project Third Runway – Embankment Fill and West MSE Wall, and Industrial Wastewater System Lagoon #3 Expansion Project On Second Public Notice
Applicant: Port of Seattle
Reference: 1996-4-02325

GeoSyntec Consultants (GeoSyntec) has been retained on behalf of the Airport Communities Coalition to provide a technical review of investigation, analysis and design relating to construction of the embankment fill and West Mechanically Stabilized Earth (MSE) Wall elements of the proposed Third Runway Expansion Project at the Seattle-Tacoma International Airport. This letter summarizes GeoSyntec's comments on these items. Additional comments are included in this letter regarding the proposed expansion of the Industrial Wastewater System Lagoon #3. Our technical review included the documents listed in Attachment A to this letter.

GeoSyntec is highly qualified to perform this review. GeoSyntec's personnel in charge of the review include Patrick C. Lucia, Ph.D., P.E., G.E., and Edward Kavazanjian, Jr., Ph.D., P.E., G.E.

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Exhibit 401
Date 2/19/02
Witness Kavazanjian
D... Mills Court Reporter

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Dr. Patrick C. Lucia is a Principal with GeoSyntec Consultants' Walnut Creek office, with over 25 years experience in geotechnical engineering. Dr. Lucia has been involved in numerous reinforced walls and slope projects and has designed reinforced walls and slopes up to 90 feet high. Dr. Lucia has served on the faculty at the University of California at Berkeley and Davis as a Visiting and Senior Lecturer respectively. He has been an invited speaker at a NATO Conference in Turkey on technology transfer with former Soviet Union countries and has lectured at Universities around the United States. He has also served as a consultant to the Panama Canal Commission on slope stability problems associated with widening of the canal.

Dr. Edward Kavazanjian, Jr., is a principal with the GeoSyntec Consultants' Huntington Beach office. Dr. Kavazanjian has extensive experience in research, practice, and education in geotechnical and environmental engineering, including fifteen years in consulting practice and seven years on the faculty at Stanford University. He is widely recognized for his work on the geotechnical aspects of earthquake engineering. Dr. Kavazanjian is lead author of the Federal Highway Administration Geotechnical Engineering Circular Number 3, *Design Guidance: Geotechnical Earthquake Engineering for Highways*. In 1999, he chaired the *Transportation Research Board Workshop on New Approaches to Liquefaction Analysis*. He served as principal investigator on the National Science Foundation sponsored joint GeoSyntec-U.C. Berkeley research project on performance of landfills in the 1994 Northridge earthquake. He chaired a session on liquefaction at the Ninth World Conference on Earthquake Engineering and delivered invited papers on the seismic design of landfills and waste containment systems at the Third International Conference of Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics and at the Eighth Canadian Conference on Earthquake Engineering. Dr. Kavazanjian currently serves as chairman of the ASCE Geo Institute Embankments, Dams, and Slopes Committee and is past chairman of the ASCE Geotechnical Division Safety and Reliability Committee. He is also a member of the Seismic Risk and Transportation Committees of the ASCE Technical Council on Lifeline Earthquake Engineering and of the Committee on Foundations for Bridges and Other Structures for the Transportation Research Board.

The GeoSyntec review of the project documents listed in Attachment A has revealed significant deficiencies in the field and laboratory investigation, and in the analysis of this project. The documents we have reviewed do not provide a sufficient basis for the conclusion that the project as conceived can withstand the static and seismic loads it will be subject to over its lifetime. The static and seismic analyses performed are not based on sound interpretation of either existing foundation conditions or the seismic conditions at the site. The analyses have not been performed in a sufficiently thorough manner or to a sufficient level of detail to deserve the approval of the U.S. Army Corps of Engineers or the Washington State Department of Ecology.

The Department of Ecology has examined the geotechnical engineering aspects of the West MSE Wall during preliminary stages of the project. In a memorandum to Mr. Tom Luster, Mr. Jerrald LaVassar of Ecology's Dam Safety Office stated "Clearly, the considerable height of the wall dictates that it be founded on a dense, unyielding foundation or a structural fill that spans between such a stratum and the base of the wall." This is not being done. Instead, a zone of weak peat and loose, liquefiable sands directly beneath the wall footprint are proposed to be densified in place, followed by construction of the tallest MSE wall in the world in a very seismically sensitive area. Mr. LeVassar acknowledged in his memo that his remarks were based on limited site specific data. We find it surprising that approval can be considered for a project of this magnitude on the basis of limited site specific data before detailed design and construction plans had been prepared. A thorough geotechnical review should be performed by the Department of Ecology in light of the numerous changes since Mr. LaVassar's last examination of the project.

Given the unprecedented scale of the West MSE Wall, this project demands the utmost in care in all aspects of investigation, analysis, and design. We are very concerned that this care has not been taken and that the resulting deficiencies could lead to a design of the embankment and walls that could ultimately result in damage or failure of the wall, particularly under the influence of a strong seismic event in the Seattle area. This could have dire consequences on both the functionality of the airport and preservation of the creek and wetlands below.

Several key points and additional concerns will be made in the discussion that follows. Of these, we wish to highlight the following:

- there is insufficient laboratory strength data for proper characterization of foundation soils, and the limited data is being interpreted incorrectly, and in an unconservative manner;

- the extent of the potentially liquefiable material may have been underestimated, and strength values being assigned to liquefied materials are unconservative;
- seismic stability analyses are being performed incorrectly;
- seismic design criteria have not been well established, and thus it is impossible to determine how the wall is intended to perform during an earthquake; and
- the FLAC analysis being performed to assess seismic performance of the wall has not been calibrated or validated with any real data, and thus it is not possible to interpret the results it provides.

The net result of these deficiencies is that the project proponent has yet to demonstrate either that a stable wall can be economically constructed or that the wall, if constructed, can withstand the seismic loads to which it may be subjected without large, unacceptable deformations.

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Comment 1: The West MSE Wall should be considered at least 153 ft high.

At its highest point, which occurs at approximately Station 180+00 in project documents, the West MSE Wall has a total exposed height of 133.5 ft, with additional embedment bringing the height of the reinforced structure to 140.3 ft. An embankment is planned above the top of the reinforced wall, raising the total height an additional 20 ft. The combined exposed height of the wall and the overlying embankment that the wall supports is approximately 153 feet. To our knowledge, a MSE wall of this height has never previously been built. Similar walls nearing this height (e.g., Tsing Yi Island wall in Hong Kong at 131 ft, Shikoku Island wall in Japan at 125 ft) have never been subjected to strong seismicity. Considering this unprecedented height and considering the strong seismicity of the Seattle area, this project demands the utmost level of care and attention to detail throughout.

Comment 2: There is insufficient laboratory testing data in the vicinity of the West MSE Wall relative to the scale of the project.

Laboratory testing summarized in the report titled "Subsurface Conditions Data Report – West MSE Wall – Third Runway Embankment – Sea-Tac International Airport" (June 2000, Hart Crowser) indicates that only seven samples have been tested for strength determination in the vicinity of the West Wall. Of those seven samples, three were tested under Consolidated Undrained (CU) conditions and four were tested under Unconsolidated Undrained conditions. Of these seven tests, three were performed at depth in the strongest subgrade materials, leaving only four tests performed in the materials most likely to be critical to slope stability concerns. Additionally, only one test (from boring HC00-B132) was performed in the vicinity of the critical wall cross-section where the wall reaches the previously discussed high point.

Given the critical nature of the project for the well being of both the airport and Miller Creek and surrounding wetlands, and the unprecedented scale of the project, which will result in construction of likely the highest MSE wall in the world, relying on this minimal level of testing is dangerous and completely inadequate. Additional borings must be performed with targeted high-quality sample collection for an expanded laboratory testing program that should focus not only on increasing the spatial distribution of testing, but should also include sufficient tests within any given soil layer to provide redundancy in the testing results and confidence in the ultimately selected strength values. This testing should additionally be used to calibrate measured strength with the results of the five cone penetration tests performed at the site in order to expand the applicability of the testing program.

It should also be pointed out that while the preceding level of testing is specific to the West MSE Wall, it is equally likely that additional testing is required for the other two MSE walls.

Comment 3: Laboratory strength test data is being interpreted in a manner resulting in higher strengths than would typically be used in engineering practice.

Results of laboratory strength tests by Hart Crowser are included in Appendix B of the "Subsurface Conditions Data Report – West MSE Wall" report (June 2000). Examination of the included CU and UU test results indicates that they are being carried out to strains on the order of 20%. Several of the materials tested do not reach a visible peak deviator stress by the end of the test, and the resulting strengths are being interpreted at the highest recorded stress, which occurs at the end of the test, at 20% strain. In conventional engineering practice, a limiting strain of 10% to 15% is normally used for interpretation of strength from laboratory results, due both to the assumptions inherent in calculation of stresses from triaxial tests (i.e. use of constant cross-sectional sample area), and to field considerations, where 10% to 15% strain in the field would typically represent a failed condition anyway. It is recommended that the testing data be reevaluated with a limit of 10% strain used for interpretation of material strengths. This will result in a reduction in the interpreted strengths for many of the tests. These reduced strengths will likely lead to lower computed factors of safety against failure (see Attachment B for a discussion of "factor of safety"), and more deformation of the wall. It is recommended that a complete reevaluation of the laboratory test data for the Third Runway project be performed, as it is likely that the deficiencies pointed out here are not specific to the West MSE Wall alone.

Comment 4: Potentially unconservative strength values are being used in stability analysis.

In addition to the potentially high strengths discussed in Comment 3, the interpreted strengths are being applied in stability analyses under stress conditions that are much greater than those tested in the laboratory. CU tests were performed in the laboratory under a maximum consolidation pressure of 12,000 pounds per square foot (psf). After placement of 160 ft or more of fill at the project site, which weighs an estimated 135 to 140 pounds per cubic foot (pcf), these materials will in fact be subjected to on the order of 24,000 psf, double the laboratory conditions. It is in fact quite common for soils to exhibit a decrease in friction angle under higher confinement, in which case the foundation soils may not be as strong as Hart Crowser is representing them, resulting in serious implications on the stability of the wall.

The ramifications of the limited test data on the stability analysis can be significant in situations where there is not much room between the computed factor of safety and the required factor of safety (see Attachment B for a discussion of "factor of safety"). For example, if a liquefaction analysis results in a factor of safety of 1.15, and the required factor of safety is 1.1, it is theoretically stable. However, if this analysis is based on a friction angle of 35 degrees in medium dense sand, while the actual friction angle at high confinement is closer to 33 degrees, the available strength in this material decreases by approximately 1200 psf, which may be sufficient to drop the factor of safety below 1.1. Such a decrease in factor of safety would indicate that the wall is not being designed with a sufficient margin of safety, which could result in excessive deformations or failure of the wall, particularly during a strong seismic event.

Given the unprecedented scale and the critical nature of the project, it is important that testing be performed to properly account for the true field conditions.

Comment 5: Flaws in the liquefaction analysis of foundation soils render the conclusion that the wall will not fail due to liquefaction invalid. Because of these flaws, the extent of potential liquefaction of the subgrade beneath the West MSE Wall and the rest of the Third Runway project may have been underestimated.

The liquefaction analysis described in the September 7, 2000 Hart Crowser memo appears to have been done primarily by statistical analysis, with little spatial analysis. The database was split up into gross subdivisions based on geometry (e.g., the West Wall, the 2H:1V embankment) but there was no evidence of further spatial analysis, e.g., looking for weak seams at a consistent elevation.

Furthermore, Hart Crowser appears to have incorrectly applied the screening criteria used to identify nonliquefiable soils. These criteria are intended to identify material that is potentially liquefiable. Inverting them to identify soils that are not liquefiable is not appropriate. Hart Crowser states, "if any one of these criteria was not met, the soil was deemed nonliquefiable." [underlining added for emphasis] The four screening criteria are:

1. (Fraction of fines finer than 0.005 mm – 5%) < 15%;
2. (Liquid limit + 1%) < 35%;
3. (Natural water content + 2%) > 0.9 LL; and
4. Liquidity index ≤ 0.75.

This is not the correct manner in which to apply these criteria. These criteria were developed for evaluation of materials that are potentially liquefiable, not for identification of materials that are not liquefiable. For instance, while soils with fines content of less than 15 percent (Criterion 1) must always be considered liquefiable, not all soils with fines content greater than 15 percent are non-liquefiable. This criterion is of particular importance in Seattle, where glacial soils may have a large percentage of "non-plastic" fines. Such soils could easily have a fines content greater than 15 percent and yet still be liquefiable, contrary to the Hart Crowser screening analysis. This inappropriate application of the screening criteria means that potentially liquefiable soils may have been identified as nonliquefiable by Hart Crowser.

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Comment 6: Inappropriate selection of residual shear strength values means that the conclusion that the wall will not slide on its foundation in the aftermath of a major earthquake is not valid. The selection of residual strength values to represent conditions after a seismic event is unconservative and some values are based upon extrapolation beyond the range of past experience.

Residual shear strengths are taken from the Seed and Harder plot as a function of SPT blow count. The mid-range of the bands drawn by Seed and Harder are used. This is not consistent with current practice, wherein the lower third to lower quartile of the band is generally used. We recommend the lower quartile. Furthermore, residual shear strength is extrapolated to blow counts of 24, well beyond the range of the Seed and Harder plot, and to values in excess of 1000 psf. The greatest observed residual shear strength on the Seed and Harder plot is 600 psf. Hart Crowser reports extrapolated values of over twice that amount, up to 1300 psf. By using values that are higher than the accepted engineering standards and outside of the range of an already limited Seed and Harder data set, the designers are taking a dangerous design step without any theoretical or experimental evidence supporting their interpretation.

Comment 7: The methodology used in performing pseudo-static (seismic) stability analysis is incorrect and may seriously underestimate the ability of the wall to withstand seismic loads.

According to Hart Crowser, "We typically apply the seismic coefficient to the most critical failure surface identified in the steady-state condition." No justification is given for using this methodology, and it is in fact incorrect as the critical static (steady-state) and seismic failure surfaces are frequently very different. Under pseudo-static conditions, a horizontal acceleration is applied to the entire failure mass, which acts as a destabilizing force. The computed critical failure surfaces for the seismic case tend to be longer, extending further back into the slope in order to collect more driving mass. The critical surface for the seismic case will also frequently extend along a weak material interface, such as the existing peat layer, or through the liquefied sand deposit.

A proper pseudo-static slope stability analysis should be performed to search for the critical failure surface independently of the static analysis. Additionally, "sliding block" failure surfaces that propagate along the weak seams should be examined, rather than just circular surfaces that cut across them. The Slope/W program that Hart Crowser is using is well suited to explore these alternate failure surfaces, and to search carefully for an independent critical pseudo-static failure surface. This analysis will likely result in a reduced factor of safety and may lead to requirements for additional ground improvement.

Figure 1 shows a conceptual sketch of a representative failure surface under pseudo-static conditions, extending through the weak peat layer far back into the fill (and potentially beyond the limits of the modeled cross-section). As currently analyzed and designed only the weak soils directly below the wall are being improved. If the critical seismic failure surface extends along the weak peat layer or liquefied zone farther back into the embankment than the static surface, the areas for ground improvement will also need to extend further back in order to remove the threat of these weak soils under a strong earthquake.

Comment 8: There are inconsistencies in the results of the Probabilistic Seismic Hazard Analysis (PSHA) performed by Hart Crowser that cast doubt on the validity of the analysis. The primary inconsistency in the PSHA is with respect to the magnitude of earthquake assigned to the various probability levels addressed in the analysis. Unless these inconsistencies are resolved, we cannot determine whether or not the design earthquake has been properly characterized.

The earthquake magnitudes assigned by Hart Crowser to the various probability levels are inconsistent with results from the United States Geological Survey (USGS) National Seismic Hazard Mapping Project and with results from analyses GeoSyntec and others have conducted for projects in the same vicinity. The progressively higher peak horizontal ground acceleration (PHGA) values associated with the progressively smaller probability levels are attributed by Hart Crowser to progressively larger magnitude "subduction zone" (offshore) earthquakes, while our work and the USGS information indicates that these higher accelerations should be associated with the local "crustal" faults (e.g., the Seattle fault). This inconsistency casts suspicion on the entire analysis. This suspicion is heightened by the observation that the Hart Crowser acceleration response spectra (curves derived from the PSHA) agree remarkably well with the USGS values, despite the fact that these curves depend primarily on earthquake magnitude. It is hard to say without further study exactly what the source of the discrepancies is. However, unless it is resolved we must consider that the seismic environment at the project site has not been properly characterized.

Comment 9: The single time history used to analyze the seismic performance of the wall does not provide an appropriate basis for the conclusion that the wall can withstand the design earthquake.

It appears that a single time history was used to characterize the design ground motions. This time history is a synthetic time history that is attributed to Steve Kramer at the University of Washington. The acceleration response spectrum for this time history is not provided. However, visual inspection indicates that this time history represents a long period (or low frequency) motion (a long, "rolling" motion) and does not contain a lot of energy at shorter periods or higher frequencies (i.e., does not contain enough "punch"). This is an important point because our analysis indicates the resonant frequency of the high wall (i.e., wall sections over 100-ft (30-m) high) is in the same relatively short frequency range where the design motion is deficient. In other words, the earthquake time history used in the analysis does not have enough energy in the range in which the wall is most sensitive to vibrations. This means that the time history used in the design analyses does not truly "test" the wall to the level of seismic force expected in the design earthquake.

Even without the above-cited frequency deficiency, we do not believe it is appropriate to use only one time history to evaluate the adequacy of the design. Given the uncertainty and variability associated with earthquake ground motions, the seismic analysis should be based on a suite of at least three or more time-histories that envelop the design acceleration response spectra.

Comment 10: Seismic design ground motion criteria have not been established and there do not appear to be any established seismic performance criteria for the wall.

The designers remain non-committal on what the seismic design ground motion level is, i.e., on the level of probability that will be used for design. While initial reports discussed ground motions with 50, 10, 5, and 2 percent probabilities of being exceeded in 50 years, later reports have discussed primarily the 10 percent (475-year return period) and

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5 percent (975-year return period) probability levels. Hart Crowser has stated, "we understand the Port of Seattle used the 475-year event for design of the South Terminal Expansion and for analysis of deepening the berths at the Terminal 5 Wharf" (April 10, 2000, Hart Crowser Memo). We do not believe the 475-year event is adequate for this project. The 475-year event (a 10 percent in 50 year design level) is the Uniform Building Code requirement for ordinary buildings, e.g. for residential construction. This project is far more important than typical residential construction.

We recommend that the "performance based design" approach be employed. In performance based design, the performance of a structure under seismic loads is defined over a broad spectrum of levels, from the load level at which no damage will occur to the load level at total collapse. Once these levels and their associated probabilities are defined, an informed decision can be made on the adequacy of the design. The earthquake engineering profession, in general, is moving towards this method of design, having recognized that this type of analysis is necessary to truly understand the adequacy of a design in a complex and uncertain seismic environment.

The designers also remain non-committal on the seismic performance criteria. The level of calculated seismic deformation in the MSE wall that is considered acceptable is never stated. In fact, the designers never even explicitly state that the MSE wall deformation that they calculate in the design event (on the order of 8 to 10 in. (200 to 250 mm)) is acceptable. The seismic performance criteria (e.g., the acceptable level of seismic deformation) for the MSE wall should be clearly stated and should be substantiated based upon the observed performance of MSE walls in earthquakes.

Comment 11: To our knowledge, the computer program FLAC used to evaluate the seismic performance of the wall in the design earthquake has never been demonstrated to reliably predict seismic deformations of earth structures. Therefore, the FLAC analyses do not provide an appropriate basis from which to conclude that the wall can withstand the design earthquake. We have additional concerns about the method of performing the analysis relating to seismic input, method of dealing with liquefaction, and residual strengths that are not properly documented in the material available for review.

FLAC was used to estimate the deformation of the MSE wall subjected to the design earthquake ground motion (the ground motion time history addressed in Comment

9). For the purpose of seismic deformation analysis of MSE structures, FLAC is at best described as unverified, and therefore unreliable. In fact, to our knowledge, there has been no demonstration of the program's ability to properly predict the seismic deformation of any type of earth structure. This type of demonstration is typically conducted by comparison of predictions made using the computer program to well-documented field observations or model tests. This deficiency is significant for conventional earth structures (e.g., soil embankments or dams) and becomes even more critical when computer modeling a reinforced earth structure due to the intricacies of modeling the reinforcement (e.g., modeling the interface elements and their behavior under cyclic loads). Certainly, for a project of this unprecedented magnitude and scope, some type of calibration exercise (e.g., comparison with centrifuge model tests) is necessary if the FLAC computer program is to be the basis for the conclusion that the wall is seismically stable.

The FLAC analyses themselves require much more documentation, even after the program is properly verified. The documentation provided to date leaves us with numerous unanswered technical questions with significant bearing on the results of the analysis. FLAC allows the user to input his own constitutive models and elements. Was this done, or were the constitutive models and elements supplied with the program used? The size of the cross-section is very small for a seismic response analysis – were transmitting boundaries used or were the boundaries rigid? Was the design motion applied directly to the base of the cross-section or was it treated as a surface motion for a "half-space" and deconvolved. How was the liquefaction deformation analysis done? When was the residual shear strength applied – at the start of the motion or sometime during the motion? Was the residual strength only applied to the soil elements that reach full liquefaction, or were elements with low factors of safety against liquefaction assumed to also mobilize their residual strength. What is the "composite" strength approach discussed in the briefing to the Technical Review Board? Was the shear strength of the sand layer simply weighted by the residual shear strength of liquefiable soils? What about the potential for continuous weak seams? Without these details, we cannot properly assess the validity of the analyses, even after the program is verified. Therefore, without these details, any conclusion that the wall can withstand the design earthquake with acceptable deformation is not valid.

Additional Concerns

Comment 12: Very "select" backfill was assumed for the wall design, with a friction angle of 37 degrees. The plan for assuring that materials selected for backfill meet the design criteria is not provided.

Design of the West MSE Wall assumes a friction angle of 37 degrees for the "select" backfill. The Hart Crowser / Reinforced Earth Company (RECo) design team state that this corresponds to a material that is "less than 5 percent fines, well compacted, and relatively well graded" (August 21, 2000, Hart Crowser Memo). As several borrow source areas to be used for the project have apparently already been explored (September 24, 1999 Hart Crowser report), it is considered prudent to test representative samples of these materials to ensure that gradation, compaction, strength and other appropriate backfill requirements can indeed be met prior to relying on the high strength value used in design. If they do not meet the design strength of 37 degrees, alternate material sources will have to be identified and tested. A plan should be provided describing the required testing of potential backfill material, as well as the construction quality assurance plan describing testing in the field during construction to ensure that the required strengths and gradations are obtained.

Comment 13: The use of Hollow Stem Auger drilling techniques for obtaining blow counts in sandy soils below the water table is not appropriate and can lead to erroneous results, particularly in loose soils (e.g. liquefiable sands).

The selected drilling technique for the majority of the field exploration program was a hollow auger with a plug at the base that prevents soil from rising up within the auger while drilling. The plug is removed prior to collection of samples and performance of standard penetration testing to determine blow counts. In many soils, and particularly in weak or loose soils (such as liquefiable sands) upon removal of the plug, the differential in water levels around the auger and inside the auger can cause soil to rise up inside the now open stem. This can lead to disturbance of the soil near the auger tip, and result in collection of disturbed samples and erroneous blow count readings. Use of a drilling technique with known limitations on such a critical project raises concerns and casts suspicion on the field investigation program and its results.

Comment 14: Plans for construction of the West MSE Wall should include instrumentation for monitoring potential deformations and stresses.

Given the unprecedented height of the West MSE Wall, it is considered prudent to plan for installation of instrumentation behind the wall face and in the backfill to monitor for deformations both during construction and at repeated intervals during the lifetime of the wall. Additional instrumentation should be considered to monitor stresses within the reinforcement strips and at the connections between these strips and facing elements. This would serve to verify the functionality of the wall both during normal operations and after any significant seismic event, providing a comparison between the theoretical and actual performance.

This point has in fact been made to the Department of Ecology previously. In a memo from Jerald LaVassar of Ecology's Dam Safety Section to Tom Luster, Mr. LaVassar states: "All parties should recognize that a wall of this height is rare. Thus, the inclusion of various monitoring devices in the wall and backfill would provide valuable confirmation that the wall is deflecting and performing in the manner anticipated by the designers both during construction and over a long and protracted service life."

Comment 15: Use of the HELP model for the estimation of groundwater and creek recharge after construction of the runway embankment may result in underestimation of subdrain capacity, leading to a potentially destabilizing buildup of water in the subdrain.

Use of the HELP model is noted briefly in the presentation to the Technical Review Board (Hart Crowser, November 16-17, 2000). The Hydrologic Evaluation of Landfill Performance (HELP) model was designed to determine leachate generation in municipal solid waste landfills. It has been shown to perform poorly in predicting maximum infiltration rates through soil covers for landfills (e.g., in predicting the performance of evapotranspirative soil covers) and thus would not be expected to provide satisfactory predictions of infiltration through a soil berm and into a drainage system.

Comment 16: The proposed Industrial Wastewater System (IWS) Lagoon #3 expansion project may need further review by the Washington State Department of Ecology Dam Safety Office.

The IWS Lagoon #3 expansion project has apparently been reviewed and approved by the Department of Ecology's Dam Safety Office. However, only limited documentation exists of the extent of the review. Among the documents provided, only one relates to review of geotechnical engineering assumptions and analyses. This document is a two page handwritten "Geotech Review" dated May 30, 2000 with initials JML. The review ends with the following statement:

> *Will need to complete our independent analysis in future. But, by inspection the current design is suitably conservative. Time constraints presently do not allow doing the full blown analysis. Again, this will be done! The project of actual building the containment berm is scheduled in 2001.*

The question remaining is whether this "full blown" analysis was in fact performed prior to approval of the plans, or whether the project was approved "by inspection" alone. No additional documentation has been provided which might clarify this matter.

Comment 17: The Port of Seattle must assess the impact of the Third Runway and infrastructure construction on the fate and transport of contaminants in the Airport Operations and Maintenance Area.

In the vicinity of the Airport Operations and Maintenance Area, known contamination exceeds MTCA cleanup levels. To our knowledge, there has been no evaluation of the impact of installation of underdrain systems and utility corridors for the Third Runway project and infrastructure construction on the fate and transport of contaminated groundwater from these existing airport operations. The general groundwater gradient leads from the vicinity of existing contamination towards the new project area and the potentially impacted creek and wetlands. Evaluation must be performed to assess the impact of new construction activities on the potential for adverse impacts on water resources including the effects of existing contamination.

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In summary, based on our review of the available documentation, there appear to be critical deficiencies in both the field and laboratory investigations performed for this project, as well as in the analysis assumptions and methodologies used. We are very concerned that these deficiencies could lead to a design of the embankment and walls that could ultimately result in significant damage or failure of the wall, particularly under the influence of a strong seismic event in the Seattle area. As such, we request on behalf of the Airport Communities Coalition that, prior to regulatory certification or approval of the proposed Third Runway Project, the applicant be required to respond to the issues raised in this letter, and that we be granted the opportunity to provide follow-up review and comment on that response.

Sincerely,

Patrick C. Lucia, Ph.D.,P.E.,G.E.
Principal

Edward Kavazanjian, Jr., Ph.D.,P.E.,G.E.
Principal

Enclosures: List of Documents Reviewed
Discussion of Factor of Safety
vitae

cc: Peter Eglick, Helsell Fetterman LLP
Kimberly Lockard, Airport Communities Coalition

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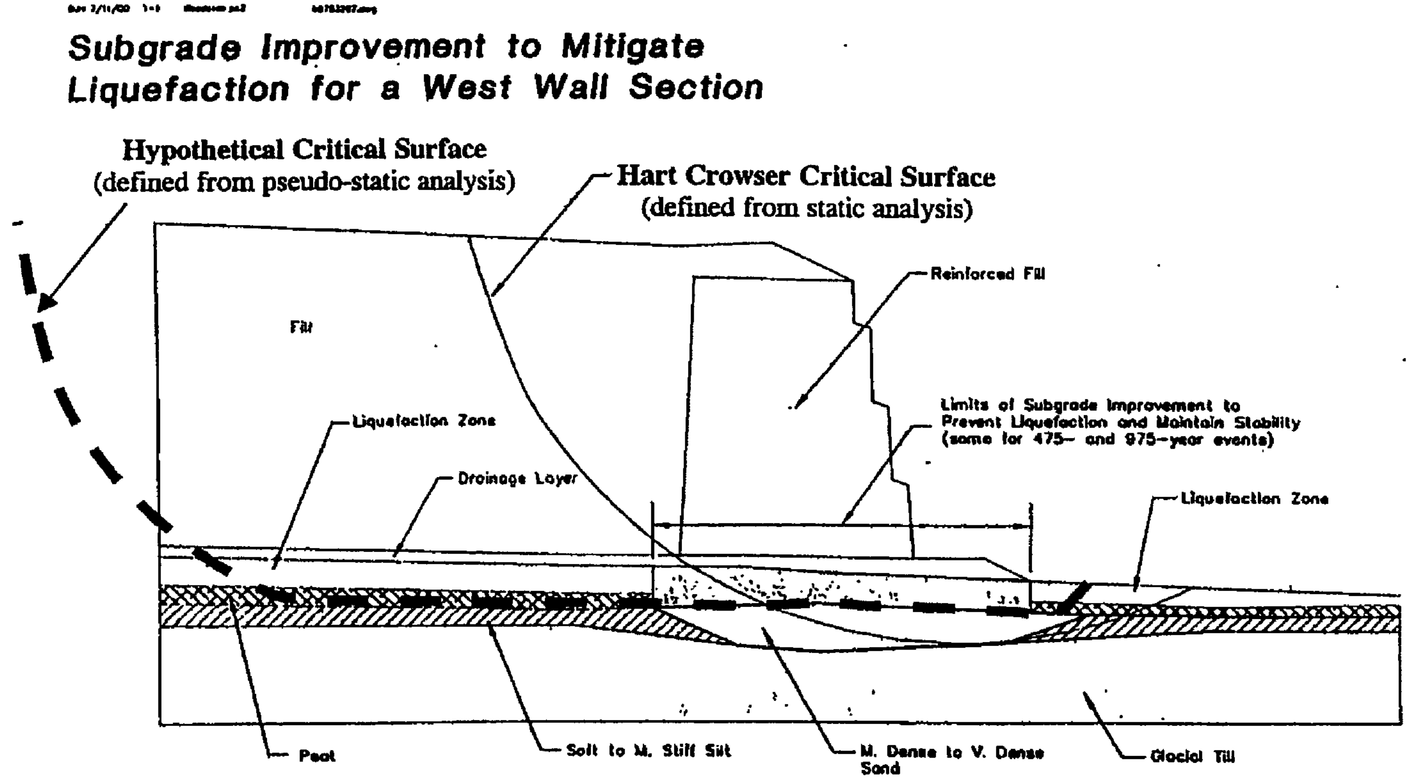

Figure 1: Hypothetical Seismic Stability Failure Surface

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Attachment A
List of Documents Reviewed

"Evaluation of Retaining Wall/Slope Alternatives to Reduce Impacts to Miller Creek – Embankment Station 174+00 to 186+00," Prepared by HNTB, Hart Crowser, Inc., and Parametrix, (No Date).

"Evaluation of Retaining Wall/Slope Alternatives to Reduce Impacts to Miller Creek – Embankment Station 174+00 to 186+00," Memorandum from Jerald LaVassar (Washington State Dept. of Ecology Dam Safety Office) to Tom Luster (Washington State Department of Ecology) regarding a review of the document in the title, (Date Unknown).

"30% Submittal – Third Runway – Embankment Construction – Phase 4," HNTB Corporation, (No Date).

"Industrial Wastewater Treatment Engineering Report," Kennedy/Jenks Consultants, December 1995 (incomplete).

"Geotechnical Design Recommendations – Phase 1 Embankment Construction – Third Runway Project – Sea-Tac International Airport – Seatac, Washington," Prepared for HNTB Corporation by AGI Technologies, January 22, 1998.

"Addendum to the IWS Engineering Report," Kennedy/Jenks Consultants, April 1998.

"Base Preparation Stability Analysis (Phase II)," Hart Crowser Memorandum, August 13, 1998.

"Approach to Stability Assessment," Hart Crowser Memorandum, August 18, 1998.

"Geotechnical Engineering Report – 404 Permit Support – Third Runway Embankment – Sea-Tac International Airport," Prepared for HNTB Corporation and The Port of Seattle by Hart Crowser, July 9, 1999.

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"Subsurface Conditions Data Report – 404 Permit Support – Third Runway Embankment," Prepared for HNTB Corporation and The Port of Seattle by Hart Crowser, July 1999.

"Subsurface Conditions Data Report – Borrow Areas 1, 3, and 4 – Sea-Tac Airport Third Runway," Prepared for HTNB and the Port of Seattle by Hart Crowser, September 24, 1999.

"Sea-Tac Airport Third Runway – Probabilistic Seismic Hazard Analysis," Hart Crowser Memorandum, October 8, 1999.

"Hydrogeologic Investigation Report – Industrial Wastewater System – Lagoon #3 Upgrade – Seattle-Tacoma International Airport," for the Port of Seattle by Kennedy/Jenks Consultants, February, 2000.

"Seattle-Tacoma International Airport – Industrial Wastewater System Lagoon #3 Expansion Project," Plan Set, Kennedy/Jenks Consultants, March 13, 2000.

"Project Manual, Including Specifications, for Industrial Wastewater system Lagoon #3 Expansion Project," Port of Seattle, March 16, 2000.

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Attachment B

Discussion of Factor of Safety

A computer program is used to evaluate the factor of safety of a given wall or slope geometry. The factor of safety represents the ratio between the strength of the soils and the forces of gravity that act on the slope. If the strength of the soil in the slope just equals the forces acting on the slope then the calculated factor of safety in the computer program will be equal to 1.0. Accepted engineering practice requires that the factor of safety be at least 1.5 under static conditions, indicating that the strength of the soils are at least 50% greater than the forces acting on the slope. This additional 50% factor of safety is intended to account for the uncertainties in the interpretation of the field and laboratory data. When evaluating the factor of safety against liquefaction during a seismic event, or under short term conditions such as construction, a reduced factor of safety is sometimes allowed. In all cases, there needs to be a margin of safety sufficient to protect against potential events, known and unknown, that could compromise the safety of the slope and lead to failure.

The computer analyses calculate the resisting strength of the soil and the destabilizing forces acting on specified potential failure surfaces within the slope. The ratio of the strength along the specified surface to the forces on that surface is then calculated as the factor of safety. There are an infinite number of surfaces within the slope for which the factor of safety can be calculated. The computer program will search within the slope to find the surface with the minimum calculated factor of safety. If artificial constraints are put into the analyses, such as preventing the computer for search for the critical seismic surface, then the program will find the minimum factor of safety only within the limits of the constrained analyses. If the analyses are improperly constrained or the slope is incorrectly modeled (e.g., with incorrect soil strengths) then the minimum factor of safety of the slope cannot be accurately evaluated.