Kavazanjian

Pre-Filed Testimony of

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

Submitted on behalf of Appellant

Airport Communities Coalition

PCHB No. 01-160

ACC & CASE v. Dept. of Ecology & Port of Seattle

February 22, 2002

TERMS OF REFERENCE

This document constitutes my pre-filed testimony in the matter of the Airport Communities Coalition (ACC) versus the State of Washington, Department of Ecology and the Port of Seattle regarding "Appeal of Section 401 Certification No. 1996-4-02325 and CZMA concurrency statement, issued August 10, 2001, related to Construction of a Third Runway and related projects at Seattle Tacoma International Airport" (the 401 Certification). This testimony is given in support of the ACC appeal of the 401 Certification by the Department of Ecology. The appendices to this document include previous comment letters submitted under my signature to the Department of Ecology and the Corps of Engineers on the Third Runway Project and various technical documents in support of this testimony.

QUALIFICATIONS

I am a civil engineer specializing in Geotechnical Engineering Analysis and Design, including Geotechnical Earthquake Engineering. I have a Bachelor of Science degree in Civil Engineering and a Master of Science degree in Civil Engineering specializing in Geotechnical Engineering from the Massachusetts Institute of Technology. I have a Doctor of Philosophy in Civil Engineering specializing in Geotechnical Engineering from the University of California at Berkeley. I am a Registered Professional (Civil) Engineer in Washington. I was an Assistant Professor at Stanford University for seven (7) years, teaching Civil and Geotechnical engineering, and have been a consulting engineer for the past 16 years. I am currently employed as a Principal at GeoSyntec Consultants (GeoSyntec).

My consulting experience includes analysis, design, and construction of mechanically stabilized earth (MSE) walls up to 40 ft in height and embankments up to 300 ft high in areas of high seismicity. I am co-author of the Federal Highway Administration (FHWA) guidance document on "Geotechnical Earthquake Engineering for Highways" and the Environmental Protection Agency (EPA) guidance document on "Subtitle D (40 CRF 258) Seismic Design Guidance for Municipal Solid Waste Landfill Facilities." I am currently lead instructor for the FHWA National Highway Institute course on Geotechnical Earthquake Engineering. I am past chairman of the American Society of Civil Engineers (ASCE) Geotechnical Division Reliability and Safety Committee and the ASCE Geo-Institute Embankments, Dams, and Slopes Committee. I currently serve on the Seismic Risk Committee of the ASCE Technical Council on Lifeline Earthquake Engineering. On February 16, 2002, I made an invited presentation on the "Proposed AASHTO Guide Specification on Seismic Design of Bridges" at the request of FHWA geotechnical engineering representatives at the ASCE Geo-Institute International Congress on Deep Foundations. I currently serve as chairman of a task force formed by the Geotechnical Group of the Los Angeles Section of the American Society of Civil Engineers to develop guidelines on design and construction of Mechanically Stabilized Earth (MSE) walls and slopes for seven building department jurisdictions in southern California (the counties of Los angles, Orange, San Diego, Riverside, San Bernardino and Ventura and the City of Los Angeles).

INTRODUCTION

My testimony focuses on the proposed West MSE wall. This MSE wall is a significant structure with an exposed face up to 135 ft high, running for a length of approximately 1500 ft, and topped by a 20-ft high sloped embankment. This is an unprecedented height for an MSE wall in an area of moderate to high seismic exposure. My testimony focuses on the following three aspects of the proposed West MSE wall.

1. The design of the West MSE wall is not complete and is still evolving. Substantial changes in design that create significant new environmental impacts have been made since the 401 Certification was issued by the Department of Ecology. Until the West MSE wall design is complete and the public has had the opportunity to review and comment on the design, the impacts of West MSE wall construction on Miller Creek and the adjacent wetlands and upon the fate and transport of contaminants that exist beneath the airport property cannot be properly evaluated and appropriate mitigation measures cannot be established.
2. The design basis earthquake loading for the West MSE wall is arbitrary and may not be appropriate. Selection of the design earthquake loading was an arbitrary decision made solely by the Port of Seattle (the Port), with no opportunity for review or comment by the public or other stakeholders, and was based on flawed logic and outdated American Association of State Highway and Transportation Officials (AASHTO) guide specifications. The seismic design criteria used by the Port is inconsistent with design criteria used on recent major transportation projects in Washington and elsewhere and was developed without consideration

of ecological risk, loss of life short of that associated with catastrophic events such as a dam failure or nuclear power plant accident, or the extended service life of an earthen embankment.

3. Conclusions regarding the seismic safety of the West MSE wall are based upon unproven, and thus unreliable, numerical analyses and upon unwarranted extrapolation from satisfactory performance of much smaller MSE walls in earthquakes. Due to the combination of unproven numerical analyses and unwarranted extrapolation from past earthquakes, the analyses performed to date do not provide reasonable assurance of satisfactory performance of the West MSE wall in the design earthquake. Even a partial failure of the West MSE wall during a seismic event could have catastrophic consequences with respect to Miller Creek and nearby wetlands. Furthermore, due to access constraints, repair of even minimal damage to the wall will likely require access with heavy equipment through the wetlands adjacent to the wall, degrading and destroying sensitive habitat.

DESIGN IS NOT COMPLETE

Only a careful reading of the latest comprehensive summary of the design of the West MSE wall, the November 2, 2001 "Geotechnical Summary Report, Third Runway Embankment and MSE Retaining Wall, Seattle-Tacoma International Airport" (the Geotechnical Summary Report) by the Port's geotechnical consultant, Hart Crowser, Inc. (Hart Crowser), makes it clear that West MSE wall design has changed significantly since the 401 Certification was issued by the Department of Ecology. In one brief paragraph, on Page 13 of the Geotechnical Summary

Report, it is noted that "the Port plans to excavate the problematic soils (generally loose saturated sands, soft to stiff silt and clay soils, and peat) and replace them with densely compacted select fill." The use of this "excavate and replace" technique is a significant departure from previous geotechnical reports and from the application for 401 Certification, which state that in situ ground improvement using "stone columns" will be employed in lieu of, excavate and replace to mitigate wetlands impacts. The only other mention of excavate and replace in the November Geotechnical Summary Report is in the Construction Control section, where it is stated that "The remove and replace method was selected because it would achieve better construction reliability." The rest of the report is conspicuously silent on this design change and makes no mention whatsoever of the associated environmental impacts.

The change from in-situ ground improvement to excavate and replace is a design change of major significance. The Port has previously represented that in-situ ground improvement would be used in lieu of excavate and replace to minimize disturbance to the wetlands and to Miller Creek adjacent to the wall. In fact, when GeoSyntec questioned whether in-situ ground improvement was feasible for the West MSE wall foundation [GeoSyntec letter of 16 February 2001 to U.S. Army Corps of Engineers and Washington State Department of Ecology], the Port vociferously rejected this suggestion, stating that "Use of the stone column technique provides a very adequate foundation that provides an alternative to making an open excavation immediately adjacent to Miller Creek and associated wetlands. This construction method avoids any potential short-term impacts associated with temporary construction dewatering" (Excerpt from Port of Seattle April 20, 2001 Responses to Public Comments).

The use of excavate and replace in lieu of in-situ ground improvement beneath the West MSE Wall may well induce significant, undocumented impacts on the adjacent wetlands and Miller Creek. The excavation for the massive structure the Port proposes to build may encroach upon Miller Creek in some locations, requiring relocation of the stream channel. In areas where the excavation will not encroach upon the creek, wetlands exist between the proposed location of the wall and Miller Creek. If an open excavation is employed, wetland soils will have to be excavated along the entire length of the subgrade improvement zones at a distance from the wall equal to two to three times the depth of the excavation. Considering a maximum excavation depth of 25 ft, excavate and replace could require removal of wetland soils over a strip of ground 75 ft wider than the previously established zone of subgrade improvement.

Degradation and disturbance to the wetlands is likely to extend over an even greater zone than the zone of excavation due to the need to dewater the excavation to facilitate compaction of the replacement soil and to maintain side slope stability. The zone in which dewatering activities drain water from, and thus degrade, the wetlands adjacent to the wall may well extend 100 ft or more from the edge of the excavation (175 ft or more from the face of the wall), depending on the method used to dewater. Wetlands within the zone affected by construction dewatering will undoubtedly be degraded, if not destroyed.

Discharge from the dewatering system represents yet another undocumented impact of the recent design change from in-situ ground improvement to excavate and replace. Depending on the size of the excavation that is dewatered at any one time and the method of dewatering, tens of thousands of gallons of water per day may be discharged at unknown locations from the excavation dewatering system. The quality of the discharged water is uncertain, as the Port has not presented any analysis of the quality of groundwater pumped from the excavation. As the quantity and quality of construction water discharges has not been analyzed and as the Port has not discussed details of how this water will be discharged, the impacts of construction water discharges are largely unknown and appropriate mitigation measures cannot be established.

Aside from the direct environmental impact to Miller Creek and the wetlands, construction dewatering may impact the fate and transport of contaminants beneath the airport operations and maintenance area and other parts of the airport property. Dewatering will draw groundwater from beneath the adjacent property towards and into the excavation. Groundwater drawn towards and into the excavation may include groundwater impacted by contaminants. Even if the impacted groundwater is not drawn into the excavation, the fate and transport of existing plumes of contaminated groundwater may be impacted by changes in groundwater flow patterns associated with dewatering. The Port has not presented any analysis of the potential impacts of excavation dewatering on the fate and transport of contaminated groundwater adjacent to the excavation.

It must be noted that, while the switch to excavate and replace mitigates concerns with respect to liquefaction of recent (Holocene) sediments directly beneath the MSE wall, liquefaction of recent sediments left in place beneath embankment fill behind the MSE wall remains a design concern. If these recent sediments liquefy and flow during and following an earthquake, they could conceivably clog the underdrain system essential to maintaining low flow to the wetlands adjacent to the wall. The Port has not presented an analysis of the potential for clogging of the underdrain by these materials should they liquefy in an earthquake.

Despite the significance of the post-401 Certification design change from in-situ ground improvement to excavate and replace beneath the West MSE Wall, this change is not mentioned in either the Executive Summary or the Conclusions of the Geotechnical Summary Report and is mentioned only briefly at two locations within the report, almost as an aside. In the balance of the Geotechnical Summary Report, the euphemism "ground improvement" is used repeatedly to refer to excavation and replacement of the unsuitable soils beneath the West MSE wall. The potentially substantial impacts of this change on Miller Creek and the adjacent wetlands, including the impacts associated with significant construction water discharges, have neither been analyzed nor even discussed by the Port. Furthermore, the Geotechnical Summary Report makes it clear that analyses of MSE wall performance are still in progress, suggesting that there may be additional unanalyzed and undisclosed impacts associated with West MSE wall construction. Until the West MSE wall design analyses are complete and all of the impacts associated with construction of the West MSE wall have been identified and evaluated appropriate mitigation measures cannot be established and Ecology cannot have reasonable assurance that water quality standards will not be violated.

THE DESIGN BASIS EARTHQUAKE IS ARBITRARY

The design basis earthquake, based upon earthquake ground motions with a 10 percent probability of being exceeded in a 50-year period (the 10% in 50-yr event), was selected solely by the Port without the involvement of other stakeholders. As this design basis earthquake was not disclosed until after the application for 401 Certification was issued, neither the public nor other stakeholders have had the opportunity to comment on it. The Port's justification for its selection of the 10% in 50-yr event as the design basis earthquake is arbitrary and flawed, as it fails to recognize significant differences between the useable life of a commercial building and the useable life of a large earth fill, is based upon outdated AASHTO guide specifications, and fails to consider the ecological risks and environmental impacts of wall failure. The relatively lax seismic design standard adopted by the Port virtually assures that the design earthquake ground motions will be exceeded over the several hundred year useful life of the proposed embankment, precluding any reasonable assurance that the design is protective of Miller Creek and the adjacent wetlands.

In the Geotechnical Summary Report, Hart Crowser states that "The Third Runway project is being designed as a 'structure of ordinary importance' similar to large public buildings and other transportation infrastructures such as bridges and highways. In technical terms, the project is designed to perform well for seismic ground motions that have a 10 percent probability of being exceeded in 50 years." The phrase "structure of ordinary importance" is an apparent reference to the Uniform Building Code, which places most commercial and residential structures in this category and specifies the 10% in 50-yr event as the minimum design standard. The Port's use of the 10% in 50-yr criteria for the Third Runway ignores the fact that the service life of the Third Runway embankment is far greater than the 50-yr service life assigned to an ordinary commercial structure. The categorization of the Third Runway as a "structure of ordinary importance" appears contrary to the assertion in the Geotechnical Summary Report that "the Port of Seattle recognizes the project is a significant engineering structure." Furthermore, the assertion that the 10% in 50-yr event used for the Third Runway project is the design event for "other transportation infrastructure such as bridges and highways" is not correct. Recent major bridge projects funded by FHWA, including the new Tacoma Narrows Bridge, have been designed to withstand earthquake ground motions with a 3 percent probability of being exceeded in 75 years, consistent with the proposed new AASHTO guide specifications discussed subsequently in this testimony.

The Port's Geotechnical Summary Report states that the 10% in 50-yr design event is appropriate because the Third Runway embankment and retaining walls "are not essential to airport operations" and "there is no risk of catastrophic loss of life due to seismic effects." These statements do not consider the environmental impacts of a seismically induced failure of the West MSE wall. Furthermore, they do not address loss of life short of catastrophic proportions, i.e., short of the loss of life, or ***in the words of the Geotechnical Summary Report,*** "such as might result from failure of a dam or nuclear power plant". Finally, considering the role of the Airport as an essential facility in post-earthquake response and recovery, it is cavalier to write off any airport facility as a structure of ordinary importance without a comprehensive analysis of the role of the airport in regional recovery. The importance of any airport facility can only be established within the context of a comprehensive evaluation of the seismic reliability of the entire airport and the impact of facility failure on the ability of the airport to meet the regions post-earthquake response and recovery needs.

The Geotechnical Summary Report makes repeated reference to AASHTO guide specifications to substantiate the design bases for the MSE wall. It is widely recognized that the AASHTO guide specifications for seismic design of bridges are obsolete and in need of revision. FHWA recently funded a major effort by the Multidisciplinary Center for Earthquake Engineering Research (MCEER), formerly the National Center for Earthquake Engineering Research (NCEER), to draft new guide specifications for seismic design of bridges. These proposed guide specifications call for evaluation of the performance of all bridges using design ground motions that have a 3 percent probability of being exceeded in 75 years, consistent with design of the new Tacoma Narrows Bridge and other recent FHWA-funded projects. The use of a 3 percent probability of exceedance in the proposed new AASHTO specifications and for these recent major projects recognizes that the 10 percent probability of exceedance used in the Uniform Building Code is not an appropriate design level for collapse of major transportation infrastructure facilities. The use of a 75-year exposure period in the proposed new AASHTO specifications and for these recent major projects recognizes the longer service life of bridges compared to buildings. The use by the Port's consultants of a 100-yr period for corrosion analyses of the reinforcing strips of the MSE wall, as stated in the Geotechnical Summary Report, is a tacit admission that the service life of the Third Runway MSE retaining walls exceeds 50 years and is at least 100 years. In fact, the useable (service) life of the embankment and runway may well be expected to be several hundred years.

In summary, use of the 10% in 50-yr design event is not consistent with either the design earthquake loading for other recent major transportation projects or proposed changes in the AASHTO guide specifications for seismic design of bridges. Selection of the 10% in 50-yr event as the design basis earthquake was based upon a flawed analogy with the design level specified in the Uniform Building Code for "ordinary structures" (e.g., commercial buildings and residential structures). This analogy fails to consider the ecological impacts of West MSE wall failure, the extended anticipated service life of the Third Runway embankment and retaining walls, the potential for non-catastrophic loss of life, or the importance of the airport to post-earthquake response and recovery efforts. The flawed decision to use the 10% in 50-yr design event was made unilaterally by the Port, without the opportunity for the public or other stakeholders to comment upon it. The Port's failure to select an appropriate seismic design event for a structure of this magnitude and importance creates substantial uncertainty over whether the West MSE wall will fail in whole or in part and thereby subject the nearby wetlands and Miller Creek to further damage and degradation.

DESIGN ANALYSES ARE UNPROVEN

The 135-ft-high free face of the West MSE wall is unprecedented for an area of moderate to high seismicity. While two other walls of similar free-face height may have performed well under static and service loads, neither of these walls has been subjected to significant seismic loading. That is to say, neither of these two walls nor, for that matter, any MSE walls even approaching this height have ever been subjected to an actual earthquake. While smaller MSE walls have, for the most part, performed well in recent earthquakes, the unprecedented height of the West MSE wall, (which is on the order of 3 times greater than walls that have been subjected to seismic loading equal to or greater than the design earthquake ground motions for the West MSE wall) renders conclusions on the seismic safety of the West MSE wall based upon performance of other MSE walls in recent earthquakes invalid.

The numerical analyses conducted by the Port's consultants using the computer program FLAC to evaluate the seismic performance of the wall must be considered unproven, and hence unreliable, as they have not been "benchmarked" (calibrated) against actual case histories or even model tests. Even if FLAC had been used successfully by other investigators to predict the performance of a similar structure subject to similar loads (it hasn't), the many options within FLAC and the complexity of the FLAC computer program mandate project-specific "benchmarking" for every new application, particularly when the project is unprecedented in scope. While the Port's Geotechnical Summary Report states that "University of Washington research demonstrates the reasonableness of FLAC seismic analysis of MSE walls," this statement is neither referenced nor supported by data.

The Geotechnical Summary Report states that "AASHTO has developed a rigorous code for design of MSE walls based on the experience of numerous state transportation agencies, other engineering organizations, and research by the Federal Highway Administration." Unfortunately, none of this experience or research involves the response of MSE walls to strong shaking in earthquakes. In fact, the City of Los Angeles Department of Building and Safety, one of the most experienced building departments in the country with respect to seismic design, does not allow the construction of MSE walls of any height within its jurisdiction due, primarily, to the lack of a seismic design methodology that the City considers to be acceptable.

For the Geotechnical Summary Report to state that "MSE walls have been used around the world, with exposed face heights of up to 140 feet" and that "this type of wall provide advantages of very good seismic performance…." is disingenuous and misleading. Neither the 137-foot-high wall built in South Africa in 1979 nor the 133-foot-high wall built in Hong Kong in 1993, cited in the Geotechnical Summary Report as tall MSE walls that "have performed well for some time" have been subjected to earthquake loading of any significance. Experience with the performance of MSE walls subject to strong shaking from earthquakes approaching the Port's design loading is limited to walls no more than half the height of the West MSE wall and is primarily with walls with a free face less than 40-ft high. It is extremely dangerous to extrapolate from the behavior of 40-ft high walls to walls with a 135-ft high free face,

AR 014151

particularly for a wall system as stiff as an MSE wall that employs steel strips (the case for the West MSE wall), as the additional height may move the resonant period of the wall into the range of periods where most of the earthquake energy is generated, maximizing the potential for structural damage. Furthermore, the focus on the "free-face height" of the wall obfuscates the importance of the sloping ground above and below the West MSE wall, which increase the effective wall height and make the West MSE wall an even-more monumental structure than the 135-ft high free face height indicates. The monumental height of the West MSE wall virtually mandates instrumentation to monitor the performance of the wall during and after construction, yet there is no discussion of the need for instrumentation in any of the Port's documents.

Of particular concern with respect to the performance of tall MSE walls are the loads on the connections between the reinforcing strips and the wall facing panels at the base of the wall. The sloping backfill behind the wall increases these loads beyond those associated with a wall of the same height with a horizontal backfill. Earthquake shaking will apply additional loads to these connections. Thus comparison between the West MSE wall and a wall with a similar free-face height but a level backfill built in a non-seismic zone is not appropriate. The connections between the reinforcing strips and the wall facing panels at the base of the wall are particularly critical items, as failure of these connections could lead to a global failure of the wall. Given its critical nature, instrumentation and monitoring of this connection are essential to verifying the accuracy of design calculations and the integrity of the wall during and after construction. In addition to relying on extrapolation from the performance of much smaller walls in earthquakes, the design team is using the computer program FLAC to evaluate the seismic performance of the West MSE wall. FLAC is undeniably a powerful and versatile numerical model that can be used to calculate the seismic response of reinforced earth structures (e.g., MSE walls). FLAC provides the user with the option of employing a variety of different sophisticated and complex constitutive models and elements to model soil, reinforcing materials, and facing panels for MSE walls. Users may also install their own material models and elements within FLAC. However, sophistication and complexity do not in and of themselves ensure that a numerical model produces accurate and reliable results. In fact, the more sophisticated and complex a model is, the more sensitive the results of analyses made using the model are likely to be to discretionary choices made in the analysis. While the raw input data for some of the Port's FLAC analyses have recently been provided to us, no report has been produced explaining the analysis and the reasons for the discretionary choices. It is a difficult and laborious process to discern these discretionary choices from the FLAC input data. Furthermore, the Port has failed to provide any insight into the logic behind these discretionary choices. Considering their importance to the design of the West MSE wall, a detailed discussion of the Port's FLAC analyses, including the discretionary choices made in developing the numerical model and interpretation of the results of the analyses, should be provided for public review and comment in the form of a report.

The Port's Geotechnical Summary Report cites a series of papers from the technical literature as evidence that FLAC produces reliable and accurate results. However, most of these papers describe FLAC analyses of unreinforced earthen embankments. It cannot be assumed the West MSE wall FLAC analyses performed by Hart Crowser produce reliable and accurate results simply because previous investigators have reported that, if properly employed, FLAC can provide reliable results, particularly if these previous investigators have not considered earth reinforcement in their FLAC analyses. For a model as complex and sophisticated as FLAC, in order to demonstrate that a particular combination of model features (element type, constitutive models) provides reliable results, the specific combination of features employed in the analysis must be "benchmarked" by comparing model predictions to observed performance of either full scale structures or scale models subject to loads similar in magnitude to the design loads.

The need for benchmarking of a numerical model is particularly acute when previous analyses using the subject numerical model either have only shown that the model is capable of predicting "earthquake-like" patterns of behavior (rather than actual earthquake response) or have not employed all of the model features that will be required for the design analyses. Both of these limiting conditions hold true for the FLAC analyses of the West MSE wall. Most of the FLAC analyses cited by the Port's consultant as evidence of the reliability of the FLAC analyses involve unreinforced earthen embankments (e.g., Inel, et al., 1993; Roth et al., 1993; Makdisi, et al., 2000). The comparisons between observed and predicted behavior presented in these papers on unreinforced earthen embankment are limited in scope and show only general agreement between calculated and observed seismic response. Furthermore, several of these investigators (Roth, et al., Makdisi, et al.) use "customized" soil models that are not available in the commercial version of FLAC. The papers that describe FLAC analyses of MSE walls (Bathurst and Hatami, 1998 and 1999) describe numerical analyses only and make no comparisons with either field or model seismic performance. In light of the limited comparisons between FLAC predictions and the observed seismic behavior of unreinforced earthen embankments and in the absence of any comparisons between FLAC predictions and the observed seismic behavior of MSE walls, the ability of FLAC to reliably evaluate the performance of the West MSE wall is unverified and must be considered unproven. The need for verification of FLAC was noted by

Bathurst and Hatami (1999), who caution in their Conclusion that "Finally, at the time of this paper, the results of FLAC models of the type reported here have not been calibrated against any physical models." These authors go on to note that "physical data that can be used to guide the selection of appropriate modeling parameters, such as damping ratio, are necessary to confirm that the results of FLAC modeling are accurate." Without this type of confirmation based upon physical data, FLAC analyses cannot be considered to provide reasonable assurance that the West MSE wall can resist the design seismic loads.

Even if previous studies were available that showed FLAC could reliably model the seismic behavior of MSE walls, insufficient information is provided in technical reports produced to date by the Port's consultants to judge the adequacy of the West MSE Wall FLAC analyses. The description of the FLAC analyses provided in the Geotechnical Summary Report and other documents produced by the Port's consultants is best described as "Trust us, we did it right." Essential details such as the constitutive model used to represent the backfill, the type of element (or elements) used to model the reinforcement, and the damping ratio used in the analyses are not even mentioned, let alone discussed in detail. The Geotechnical Summary Report notes that a report on seismic design of the wall is in preparation. While this may provide some of the missing details, it is not available for scrutiny nor was it available to the Department of Ecology prior to issuance of the 401 Certification. Not only does this preclude our review and comment, it also suggests that wall design is not complete and thus that all of the impacts of wall construction may not yet be known.

CONCLUSION

The design of the West MSE Wall for the Seattle-Tacoma International Airport Third Runway Project is not yet complete and is still evolving. Recent design changes, made after 401 Certification was issued by the Department of Ecology, will result in substantial environmental impacts to Miller Creek and adjacent wetlands that were not discussed in the application for 401 Certification and have not been analyzed. These environmental impacts include wetlands destruction, wetlands degradation, and construction water discharge. Design changes resulting in significant additional environmental impacts remain a possibility until the design of the well is complete. Until all of the significant environmental impacts associated with wall construction have been identified, the public cannot comment upon them and the Department of Ecology cannot accurately assess whether appropriate mitigation measures have been established.

The Port arbitrarily selected the design basis earthquake without the opportunity for public review and comment. The rationale used to select the design earthquake was flawed in that it fails to consider the environmental impact of a wall failure, the relatively long service life of the Third Runway embankment and retaining walls (compared to commercial buildings), current practice for major transportation facilities (e.g., the new Tacoma Narrows Bridge), proposed changes to AASHTO guide specifications, loss of life short of that associated with catastrophic dam failures or nuclear power plants accidents, and the importance of the airport to regional post-earthquake response and recovery. This flawed rationale results in a design earthquake loading that is inadequate and thus does not provide reasonable assurance that the wall will not fail in an earthquake with catastrophic consequences to nearby wetlands and Miller Creek.

Conclusions regarding the seismic performance of the wall are based upon unwarranted extrapolation from the observed satisfactory performance of much smaller MSE walls subjected to strong earthquake shaking and an unverified FLAC numerical model. Experience with the performance of MSE walls in earthquakes that approached the intensity of the design earthquake is limited to walls less than half the height and typically less than one-third the height of the West MSE wall. While FLAC is a sophisticated and complex model, its ability to reliably predict the behavior of either actual MSE walls in earthquakes or model MSE walls subject to simulated seismic loads has never been demonstrated. Without this "benchmarking," the results of the FLAC analyses cannot be relied upon. Furthermore, details of the FLAC analyses have never been provided for review and comment. The combination of unwarranted extrapolation and an unproven numerical model do not provide reasonable assurance that West MSE wall can withstand the design earthquake loading.

Given the above considerations, the Port has failed to establish the true extent of impacts to the wetlands and Miller Creek from the West MSE wall. Unless and until the Port provides a proper seismic assessment of the massive MSE structure and proper assessment of the impacts of excavation and dewatering, and until the design is complete so that all other impacts of wall construction may be identified and evaluated, the Department of Ecology cannot be reasonably assured that the wetlands and streams will not suffer substantial harm from the construction and from the performance of the structure itself.

Respectfully submitted,

Edward Kavazanjian, Jr, Ph.D., P.E
Registered Professional (Civil) Engineer No. 34612

Pre-Filed Testimony
of
Dr. Ed Kavazanjian

INDEX TO EXHIBITS

A. Curriculum Vitae of Dr. Ed Kavazanjian

B. HartCrowser report dated November 2, 2001, "Geotechnical Summary Report Third Runway Emankment and MSE Retaining Walls, Seattle-Tacoma International Airport"

C. February 16, 2001 letter from GeoSyntec to the Army Corps of Engineers; comments on the Embankment Fill and West MSE Wall

D. Cover page and pages III-28 – III-42 of April 2001 responses to comments made by GeoSyntec Consultants February 16, 2001 letter.

A

EDWARD KAVAZANJIAN, JR.
Principal

earthquake engineering
geotechnical engineering
ground improvement
landfill engineering
environmental engineering
research and development

EDUCATION

University of California, Berkeley: Ph.D., Geotechnical Engineering, 1978
Massachusetts Institute of Technology: SM, Geotechnical Engineering, 1975
Massachusetts Institute of Technology: SB, Civil Engineering, 1973

PROFESSIONAL REGISTRATION

Registered Civil Engineer, Arizona, No. 28043
Registered Professional Engineer, California, No. C031834
Registered Geotechnical Engineer, California, No. GE002103
Registered Professional Engineer, Washington, No. 34612

PROFESSIONAL HISTORY

GeoSyntec Consultants, Huntington Beach, California
Principal, 1995-Present
Associate, 1992-1995
MAA Engineering Consultants, Inc., Los Angeles, California
Executive Vice President, 1990-1992
The Earth Technology Corporation, Long Beach, California
Associate, 1988-1990
Parsons, Brinckerhoff Quade and Douglas, Inc., New York, New York.
Lead Geotechnical Engineer, 1985-1987
Supervising Geotechnical Engineer, 1987-1988

Department of Civil Engineering, Stanford University, Stanford, California
Assistant Professor, 1978-1985
Department of Civil Engineering, University California, Berkeley, California
Research Assistant, 1975-1978
Department of Civil Engineering, Massachusetts Institute of Technology, Cambridge, Massachusetts. Research Assistant, 1973-1975

REPRESENTATIVE EXPERIENCE

Earthquake Engineering

Dr. Kavazanjian has extensive international experience in seismic analysis and design. He has served as lead engineer for seismic design on major infrastructure development projects in the United States and abroad and as principal investigator on federally funded earthquake hazard mitigation research projects. In addition to serving as co-author of the FHWA design guidance document on geotechnical earthquake engineering and the USEPA guidance document on seismic design of municipal solid waste landfills, Dr. Kavazanjian has authored and co-authored numerous papers on geotechnical aspects of earthquake engineering in refereed journals and conference proceedings, served on National Science Foundation review panels for earthquake hazard mitigation projects, and co-chaired a session on liquefaction at the Ninth World Conference on Earthquake Engineering. Dr. Kavazanjian is the Lead Instructor for the FHWA National Highway Institute Training Course on Geotechnical Earthquake Engineering. . He currently serves on the Seismic Risk and Transportation Committees of the ASCE Technical Council of Lifeline Earthquake Engineering. He served on the Organizing Committee for the Port of Los Angeles Seismic Workshop as co-chairman of the Risk Sub-Committee. He addressed the 1992 annual meeting of the Transportation Research Board on geotechnical aspects of seismic design for highway systems and the 1990 Southwest Regional Conference of the Society of American Military Engineers on geo-aspects of seismic design. In January 1999, Dr. Kavazanjian was responsible for organizing the workshop on "New Approaches to Liquefaction Analysis" at the 78th Annual Meeting of the Transportation Research Board. In June

1999, he was a keynote speaker on "Seismic Design of Solid Waste Landfills" at the 8th Canadian Conference of Earthquake Engineering. . In February 2002, he addressed the ASCE Geo-Institute International Congress on Deep Foundations on *Proposed AASHTO Guide LRFD Specifications for Seismic Design of Highway Bridge Foundations.*

Dr. Kavazanjian's research work on earthquake engineering includes pore pressure development during seismic loading, seismic slope stability and slope deformation analyses, liquefaction potential mapping, seismic safety of dams, probability and reliability applied to geotechnical aspects of earthquake engineering, and frictional base isolation using geosynthetic materials. Dr. Kavazanjian has served as principal investigator on a USGS sponsored project for liquefaction potential mapping of downtown San Francisco. He was also principal investigator on NSF sponsored research on pore pressure development during non-uniform loading, non-stationary random vibration theory for site response analyses, the seismic stability and deformation of infinite slopes, and the use of geosynthetics for base isolation of structures. He has served as a consultant to the U.S. Army Engineers Waterways Experiment Station on reliability and probability applied to geotechnical problems and to the Federal Emergency Management Agency on probabilistic evaluation of the seismic safety of earth dams.

Dr. Kavazanjian's consulting experience includes seismic hazard studies, seismic performance analyses, and risk and reliability analysis. He has been the engineer in responsible charge for numerous strong shaking seismic hazard studies in Southern California, including the Badger Avenue Bridge Rehabilitation and the Pier 300 Container Wharf for the Port of Los Angeles, the Harbor Generating Station Repowering Project for the Los Angeles Department of Water and Power, seismic retrofit of bridges on the State Street line for the Southern California Regional Rail Authority (SCRRA) Commuter Rail system, the Bolo Station Waste-By-Rail Landfill project in the Mojave Desert, and the rehabilitation of the B-Street Pier in San Diego. He has directed assessments of displacement potential for active faults for the Pier 300 project and at Alquist-Priolo special study zone sites for the proposed SCRRA Simi Valley Commuter Rail station and for the Alamitos Bay Development in Long Beach.

He provided senior technical review of the seismic hazard assessment for the proposed EuroRoute English Channel crossing, for the Lake Gaston Water Supply Pipeline in Virginia and for the Eagle Mountain Landfill in Riverside, California.

Dr. Kavazanjian's design experience includes seismic retrofit of over 20 bridges for Caltrans and SCRRA in Sacramento, San Diego, and Los Angeles. He has was responsible for seismic deformation analyses for the Pier 300 retaining dikes in the Port of Los Angeles and for foundation performance analyses for the Talmadge Bridge Cable-Stayed Replacement structure in Savannah, Georgia and the 60-meter diameter gravity caissons for the proposed Rion-Antirion bridge crossing in Greece. He helped develop seismic design guide specifications for high rockfill embankments for the Ankara Motorway and performed preliminary seismic design for the Istanbul Metro sunken tube crossing of the Bosporous in Turkey. He has managed numerous liquefaction potential and site response analyses in Northern and Southern California. Dr. Kavazanjian also served as geotechnical consultant for a comprehensive multi-hazard analysis of Ogden, Utah (including earthquake, flood, and debris flows) and directed the seismic risk assessment for the state of Alaska Supreme Courthouse Expansion project.

Geotechnical Engineering

Dr. Kavazanjian has extensive experience in both research and practice in geotechnical engineering. His major project experience includes investigation, design, and construction management services for highway and mass transit systems, water resource developments, port and harbor structures, and residential, commercial, and industrial development. He is recognized for his research on the behavior of soft clay soils, probability and reliability theory applied to geotechnical problems, soil improvement, geotechnical aspects of earthquake engineering, and underground construction. Dr. Kavazanjian is co-author of the Federal Highway Administration (FHWA) design guidance document on geotechnical earthquake engineering. He delivered a keynote address at the International Conference of Rheology and Soil Mechanics, chaired a session on liquefaction at the Ninth World Conference on Earthquake Engineering, and served on the organizing committee and as co-chairman of the risk sub-committee for the Port of Los Angeles Seismic Workshop. He is past chairman of the ASCE Geotechnical Division Safety and Reliability Committee and the Geo-Institute Embankments, Dams and Slopes Committee. He currently serves as chairman of the ASCE Geo-Institute Technical Coordination Council, on the Seismic Risk and Transportation Committees of the ASCE Technical Council on Lifeline Earthquake Engineering, and on the Committee on Foundations for Bridges and Other Structures for the Transportation Research Board.

In transportation engineering, Dr. Kavazanjian's experience includes bridges, tunnels, highways, and rail transit systems. He served as project manager for geotechnical design services for Caltrans seismic retrofit projects. He managed geotechnical services for design of the Melinda Road Overcrossing and Los Alisos Undercrossing on the Foothill Transportation Corridor in Orange County. Dr. Kavazanjian was geotechnical consultant for final design for the Badger Avenue Bridge in Wilmington California. For the Southern California Regional Rail Authority (SCRRA), he managed geotechnical design and construction services for the commuter rail maintenance facility at Taylor Yard in Los Angeles and directed geotechnical investigations and analyses for the San Gabriel fly-over embankment and bridge structure. He also managed geotechnical services for the Kearney Connection fly-over embankment in Kearney, New Jersey, Aviation Parkway in Tucson, Arizona, and the approaches to the Second Elizabeth River Tunnel in Norfolk, Virginia.

Dr. Kavazanjian's underground construction experience includes the environmental assessment and preliminary design for the Sepulveda Tunnel under the runways at Los Angeles International Airport. He was also responsible for final geotechnical design for the Vermont-Santa Monica Metro Rail station in Los Angeles. He managed geotechnical investigations for the Aviation Corridor drainage tunnel in Tucson, Arizona and the Superconducting Super Collider in Texas. Dr. Kavazanjian directed numerical analysis of the PATH tubes under the Hudson River and the Midtown Tunnel under the East River in New York for the Port Authority of New York and New Jersey. He also conducted geotechnical analyses for the Detroit-Windsor Tunnel between the U.S. and Canada, the Niagara Power Expansion Project in New York, the Second

Elizabeth River Crossing in Norfolk, Virginia, the proposed EuroRoute English Channel crossing, and the Post Office Square project in Boston.

Dr. Kavazanjian's Port and Harbor experience includes work on pile-supported decks, marginal wharves, dredging plans, and backlands development in New York, New Jersey, Newport News, Los Angeles and San Diego. For the Port Authority of New York and New Jersey he directed, geotechnical analyses for the capacity evaluation and rehabilitation of Pier 44 on the Hudson River and design analyses for a tieback bulkhead on the Passaic River. At the Newport News Naval Shipyard, he evaluated pile load test results, investigated the failure of a construction cofferdam, and developed a revised dredging plan for the Trident Land Level Ship Building Facilities. In Los Angeles, Dr. Kavazanjian directed the surcharging of the hydraulic fill for the Pier 300 42-Acre Site Ground Modification project. He was also responsible for the seismic hazard analyses, the Palos Verdes fault rupture potential assessment, the geotechnical investigation, and the seismic deformation analyses of retaining dikes for the Pier 300 Container Wharf Design project. He was lead geotechnical engineer for geotechnical services during construction of Firestation 111, including the pile-supported deck, dredging, and underwater fill and rip-rap placement. For the Port of Long Beach, Dr. Kavazanjian managed the geotechnical investigation and analysis for the Pier G Bulk Handling Facility. His San Diego experience includes seismic hazard and geotechnical analyses for the B-Street Pier Rehabilitation and the tunnel feasibility assessment and liquefaction analysis for the Second Harbor Entrance Geotechnical Feasibility Study.

In water resource development, Dr. Kavazanjian has worked on dam rehabilitation, sewage treatment plant expansion, storm drain and water supply pipeline design, and safety analysis of existing dams. Dr. Kavazanjian managed investigation, design, and construction for the rehabilitation of Trout Run Dam in Boyertown, Pennsylvania, including the relining of the low level outlet conduit, installation of a low permeability membrane and slurry wall cutoff upstream and prefabricated bench drains downstream, and rehabilitation of the downstream toe drain. On the Hyperion Treatment Plant Expansion in El Segundo, California, Dr. Kavazanjian directed the construction dewatering assessment for the full Secondary Digester project. He was also responsible for design of storm drains and outfalls for the Port Authority of New York and New Jersey PATH maintenance facility in Kearney, New Jersey, and for the geotechnical investigation for storm drains in the City of Diamond Bar.

Ground Improvement

Dr. Kavazanjian's experience on ground improvement projects includes design and construction of reinforced earth walls and slopes, mini-piles, stone columns, deep dynamic compaction, prefabricated vertical drains and surcharge fills, vacuum-induced consolidation of cohesive soil, and grid cell reinforcement of soft subgrades. Dr. Kavazanjian's experience with reinforced earth walls and slopes includes static and seismic design and construction of reinforced earth walls and slopes for the Operating Industries Inc. and McColl Superfund sites, design and construction of a soil nailed wall at the Sunshine Canyon landfill, and third-party review of the static and seismic design of proposed reinforced earth walls for the Vista Pacific project for the Culver City Department of Public Works. At the McColl Superfund site, the reinforced earth walls and slopes were used to enhance the stability of pits containing hazardous refinery wastes dating from World War II. At OII, Dr. Kavazanjian directed design of a geogrid reinforcement system for veneer stability of the final cover on the steep slopes immediately adjacent to a major freeway. For Culver City, Dr. Kavazanjian directed third party review of the design of reinforced earth walls up to 50-ft.tall when community groups questioned the design proposed by a developed. Dr. Kavazanjian is currently chairman of a Los Angeles Section ASCE Geotechnical Group task force developing guidelines for design and construction of reinforced earth walls for seven building department jurisdictions in southern California.

On the Southern California Regional Rail Authority (SCRRA) Commuter Rail project, Dr. Kavazanjian directed design and construction of the stone column supported reinforced earth embankment for the 4000 foot San Gabriel Flyover viaduct. He also managed design and construction of the SCRRA Taylor Yard Maintenance Facility in Los Angeles, including deep dynamic compaction of the building footprint for densification of uncertified fill and potentially liquefiable soil. For rehabilitation of the

Coney Island Maintenance Facility for the New York Transit Authority, he developed contract specifications and drawings for 30- and 40-ton mini-piles ("root" piles) installed inside the active shop area and monitored compressive and tensile load tests on production piles. For the rehabilitation of Trout Run Dam in Boyertown, Pennsylvania, he directed grouting from within a 30-in. diameter low level outlet conduit 110 ft beneath the crest and installation of a low permeability membrane and bentonite-cement cut-off wall upstream for seepage control. For the U.S. Army Engineers Waterways Experiment Station, Dr. Kavazanjian conducted laboratory testing and numerical analysis of grid cell reinforcement for pavement subgrade stabilization.

Dr. Kavazanjian was responsible for design and/or construction of surcharge fills for the Second Elizabeth River Tunnel in Newport News, Virginia, the PATH Main Repair Facility in Kearney, New Jersey, the Pier 300 42-acre site in the Port of Los Angeles, and the Port of Long Beach Pier G Bulk Handling Facility. At Newport News and the Port of Los Angeles, prefabricated vertical drains were installed to accelerate surcharge settlement. Work at the Pier 300 site included design, construction, and monitoring of a vacuum-induced consolidation test section using vertical drains capped by a sand blanket and impervious membrane.

Landfill Engineering

Dr. Kavazanjian is nationally and internationally recognized for his work on static and seismic stability analysis and seismic design of solid waste landfills. Dr. Kavazanjian is co-author of the USEPA municipal solid waste landfill seismic design guidance manual for Subtitle D compliance. He has been responsible for static and seismic analysis of numerous municipal solid waste landfills for compliance with state and federal regulations, including landfills in the states of Washington, Virginia, New York, Tennessee, and South Carolina as well as in northern and southern California. He was principal investigator for the National Science Foundation-sponsored joint GeoSyntec-University of California at Berkeley investigation of the performance of solid waste landfills in the Northridge earthquake of 17 January 1994. Dr. Kavazanjian co-chaired a 1993 National Science Foundation workshop on seismic design of solid waste landfills and served as principal investigator for a National Science Foundation-sponsored research project on measurement of shear wave velocity at municipal solid waste landfills. Dr. Kavazanjian authored the summary report on "Geotechnics of Solid Waste Landfills" for the 2nd International Congress on Environmental Geotechnics in Osaka, Japan in October 1996 and was a keynote speaker on "Seismic Design of Solid Waste Landfills" at the 8th Canadian Conference of Earthquake Engineering in June 1999.

Over the past 10 years, Dr. Kavazanjian has been extensively involved in the design and construction of municipal solid waste landfills in southern California. He has served as GeoSyntec Consultants project manager for engineering support services at the City of Los Angeles Lopez Canyon Landfill since 1993. At Lopez Canyon, he was responsible for permitting, design, construction management, and quality assurance services for development of Disposal Area C, including installation development of alternative designs for both the side slope liner and final cover systems. Design of the Area C side slope liner system included preparation of the application submitted to the Regional Water Quality Control Board of this first ever alternative to the RCRA Subtitle D prescriptive liner system approved in California. Dr. Kavazanjian's work at Lopez Canyon has also included design and construction services for the partial closure of Disposal Areas A and B, community relations, landfill gas, noise, and groundwater monitoring, and support for preparation of California Environmental Quality Act (CEQA) documents for landfill closure. His current work at Lopez Canyon includes design and construction services for closure of the landfill, including permitting and construction of an evapotranspirative soil cover for the unlined areas.

At the Sunshine Canyon Landfill, Dr. Kavazanjian has been involved in design and construction of the County Extension Landfill, closure of the City Landfill, and permitting for the City/County Landfill. For the new County Extension Landfill, his responsibilities have included master planning, Phase I design and construction, and Phase II design. For the inactive City Landfill, he has directed revision of the closure plan in response to agency comments and closure design, including surface water control facilities and evapotranspirative soil cover. For the proposed City/County Landfill, Dr. Kavazanjian provided preliminary design and engineering support for preparation of the Subsequent Environmental Impact Report (SEIR) and is directing preparation of the permit documents.

At the Shafter-Wasco and Bena Landfills in Kern County, California, Dr. Kavazanjian managed construction quality assurance services during liner construction and provided technical support on an as-needed basis. At Bena, he was also a member of the Master Planning team for landfill expansion, directing GeoSyntec support services for Master Plan preparation. Following preparation of the Bena Master Plan, Dr. Kavazanjian served as technical director for design of Phase 1 of the expansion, including preparation of the liner performance demonstration for the Central Valley Regional Water Quality Control Board (RWQCB), the first such demonstration approved by the Central Valley RWQCB. Dr. Kavazanjian was project director for preparation of the closure plan and closure design drawings and quality assurance service during construction for closure of Kern County's Lebec Landfill, including geosynthetic liner – based final cover and passive landfill gas venting system.

Dr. Kavazanjian's southern California municipal solid waste landfill experience includes the Azusa, Puente Hills, Spadra, Calabasas, and Chiquita Canyon Landfills in Los Angeles, the Olinda Alpha and Frank R. Bowerman Landfills in Orange County, the Heaps Peak, Newberry, Lucerne, Milliken, and Yucaipa Landfills in San Bernardino County, and the Badlands and Eagle Mountain Landfills in Riverside County. At Azusa, he was responsible for revisions to the Report of Disposal Site Information and design of the Zone II side slope liner extension and he preparation of the Partial Closure Plan for Zone I. The work at Azusa included development of a revised final cover plan to maximize waste capacity, landfill capacity calculations, stability analyses, design of an evapotranspirative soil cover, and drainage design. Dr. Kavazanjian was responsible for grading design at the Puente Hills and Spadra landfills, including geotechnical and geological investigations, slope stability analyses, and design and construction of a slope stabilization system using high capacity ground anchors. For the Olinda Alpha Landfill in Orange County, California, Dr. Kavazanjian managed side-slope liner design for the back-canyon area, including subsurface investigation, geological mapping, landslide remediation, preparation of a petition for an alternative liner design, and preparation of liner construction documents and a construction quality assurance plan. For the

Calabasas and Frank R. Bowerman landfills, Dr. Kavazanjian was Project Director for construction quality assurance services for composite liner construction.

At the Yucaipa and Heaps Peak Landfills, Dr. Kavazanjian directed slope stability analyses in support of final closure plan preparation. At the Newberry Springs, Lucerne, and Chiquita Canyon Landfills, Dr. Kavazanjian either managed or provided peer review for final closure plan preparation. He managed design of a modular block reinforced earth wall for closure construction at the Milliken Landfill. For the Badlands and Eagle Mountain Landfills Dr. Kavazanjian provided technical oversight for the seismic hazard and static and seismic slope stability analyses, including seismic site response and deformation analyses He also participated in preparation of the revised Environmental Impact Report (EIR) for the Eagle Mountain project.

Dr. Kavazanjian also has extensive experience with closure design and construction for hazardous waste landfills. He was project manager for preliminary design, including the geotechnical investigation, conceptual design of the containment system, and chemical compatibility testing, and provided senior technical oversight for closure design and construction at McColl Superfund site in Fullerton, California. Dr. Kavazanjian was project manager for pre-design seismic studies under Consent Decree Number 3 (CD-3) for the OII Landfill Superfund site in Monterey Park, California. The pre-design studies at OII included a geophysical investigation, large diameter bucket auger borings, design and construction of an on-site laboratory for static and dynamic soil testing, large test trench, in-situ density evaluation, seismic hazard assessment, and static and dynamic finite element analyses of the waste mass. He is currently providing technical oversight for final cover design, including remedial slope stabilization and infiltration analyses, at the OII site under CD-3. Dr. Kavazanjian directed seismic analysis for the five landfills at the Casmalia site near Santa Maria, California, including field measurement of shear wave velocity, seismic response analyses, and seismic deformation analyses.

Dr. Kavazanjian was engineer in responsible charge for stabilization of the final cover for a Cement Kiln Dust pile, restoration of the borrow area, and the Department of Transportation Deck Extension in Metalline Falls, Washington. Dr. Kavazanjian's Superfund experience includes static and seismic stability analysis of fine, compressible tailings and design of bank stabilization measures for closure of the Big River Mine Tailings site in Desloge, Missouri, and design review of the geosynthetic cover system for closure of the Hardage site in Criner, Oklahoma. Dr. Kavazanjian's hazardous waste landfill experience also includes geotechnical analyses for a proposed low-level radioactive waste disposal facility in Martinsville, Ilinois. He was also responsible for seismic analyses for the mixed waste on-site landfill at the Fernald site in Ohio.

Environmental Engineering

Dr. Kavazanjian's experience in environmental engineering also includes site characterization, remediation, and design for landfills, transportation projects, and water resource developments. He was project manager for the Phase I environmental site assessment for the Alameda Transportation Corridor, a proposed 17-mile dedicated below-grade rail and truck corridor along an existing rail right of way from downtown Los Angeles to the harbor area. He served as project manager for the Phase I environmental assessment and Phase II environmental sampling and testing for the widening of the Sepulveda Tunnel under the runways at Los Angeles International Airport (LAX). He also managed evaluation of an abandoned bio-remediation farm for the Remote Aircraft Parking Facilities Expansion at LAX. Other waste management projects in which Dr. Kavazanjian has been involved include preliminary design for the Illinois Low-Level Radioactive Waste Repository in Martinsville, Illinois and research on probabilistic analyses of toxic and hazardous waste problems for the U.S. Army Corps of Engineers Waterways Experiment Station.

Research and Development

Dr. Kavazanjian has served as principal investigator on sponsored research projects for the National Science Foundation (NSF), the United States Geological Survey (USGS), the Federal Emergency Management Agency (FEMA), the United States Department of Transportation, and the U.S. Army Corps of Engineers. For the NSF, Dr. Kavazanjian recently completed a study of the performance of solid waste landfills in the 17 January

1994 Northridge earthquake. In 1993, he was principal investigator for a NSF-sponsored study of the shear wave velocity of solid waste landfills using Spectral Analysis of Surface Waves and was co-organizer of an NSF-sponsored workshop on research needs for the seismic design of solid waste landfills. He has also served as principal investigator on Development of a Numerical Method for the Time Dependent Behavior of Soft Clay and on Probabilistic Assessment of Pore Pressure Development During Seismic Loading for NSF.

Dr. Kavazanjian developed a liquefaction map for downtown San Francisco under the USGS National Earthquake Hazard Mitigation Program. For the Department of Transportation, he was co-principal investigator for Development of a Methodology for Analyses of Advanced Technology for Soft Ground Tunneling. He served as the principal geotechnical investigator for the development of probabilistic analyses for screening and detailed hazard analyses of earth dams for FEMA. Under the auspices of the U.S. Army Engineers Waterways Experiment Station, Dr. Kavazanjian studied Application of Probability and Reliability to Geotechnical Practice and Probabilistic Analyses for Toxic and Hazardous Waste Problems.

Dr. Kavazanjian is co-author of the USEPA guidance document on seismic design of municipal solid waste landfills and of the FHWA guidance document on geotechnical earthquake engineering.

PROFESSIONAL AFFILIATIONS

- American Society of Civil Engineers (ASCE)
- International Society of Soil Mechanics and Foundation Engineers (ISSMFE)
- Solid Waste Association of North America (SWANA)
- Unites States Society on Dams (USSD)
- Earthquake Engineering Research Institute (EERI)

LIST OF PUBLICATIONS

Mitchell, J.K., Vanmarcke, E.H., Kavazanjian, E. Jr., and Hawkes, W.D. (1974), "Settlement Predictions for a Heavy Earthfill Load in South Florida," Research Project R74-43, Department of Civil Engineering, Massachusetts Institute of Technology, Cambridge, Massachusetts.

Kavazanjian, E. Jr. and Mitchell, J.K. (1977), "A General Stress-Strain-Time Formulation for Soils," Proc. *Specialty Session No. 9 on Constitutive Equations of Soils,* 9th International Conference on Soil Mechanics and Foundation Engineering, Tokyo, pp 113-120.

Kavazanjian, E. Jr., Mitchell, J.K. (1977), "Stand-Up Time of Tunnels in Squeezing Ground; Part II: A General Constitutive Relationship," *U.S. Department of Transportation Final Report under Contract No. DOT-OS-50108.*

Mitchell, J.K., Kao, T.C. and Kavazanjian, E. Jr. (1979), "Analysis of Grid Cell Reinforced Pavement Bases," *Technical Report GL-79-8*, U.S. Army Engineer Waterways Experiment Station.

Kavazanjian, E. Jr. and Hadj-Hamou, T. (1980), "Prediction of the Dynamic Material Properties of Soil from the Results of Static Shear Tests," *John A. Blume Earthquake Engineering Center Technical Report No. 45*, Stanford University.

Kavazanjian, E. Jr. and Chameau, J.L. (1981), "Effective Stress Analysis of Seismically Induced Stability Problems," Proc. *International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics*, St. Louis, Missouri.

Kavazanjian, E. Jr. and Hadj-Hamou, T. (1981), "Determination of the Dynamic Material Properties of Soils from the Results of Static Shear Tests," Proc. *International Conference on Recent Advances in Geotechnical Engineering and Soil Dynamics,* St. Louis, Missouri.

Kavazanjian, E. Jr., Roth, R.A., and Echezuria, H. (1983), "Probabilistic Evaluation of Liquefaction Potential for Downtown San Francisco," *John Blume Earthquake Engineering Center Technical Report No. 60*, Stanford University.

Kavazanjian, E. Jr., Chameau, J.L., Clough, G.W., and Hadj-Hamou, T.A. (1983), "Applications of Probabilistic Models to Geotechnical Practice," Proc. *Seminar on Probabilistic Methods in Geotechnical Engineering*, M.E.. Hynes-Griffin and L.L. Buege, editors, U.S. Army Engineers Waterways Experiment Station, Vicksburg, Mississippi.

Kavazanjian, E. Jr. and Mitchell, J.K. (1984), "The Time Dependence of Lateral Earth Pressures," *Journal of the Geotechnical Engineering Division*, American Society of Civil Engineers, Vol. 110, No. SM. 4.

Borja, R.I., and Kavazanjian, E., Jr. (1984), "Finite Element Analysis of Time-Dependent Behavior of Soft Clays," *Geotechnical Engineering Research Report GT-1*, Department of Civil Engineering, Stanford University.

Kavazanjian, E. Jr. and Ho, C.L. (1984), "Non-Linear Probabilistic Evaluation of the Number of Equivalent Uniform Cycles for Liquefaction Analyses," Proc. *8th World Conference on Earthquake Engineering*, Vol. III, San Francisco, California.

Hadj-Hamou, T. and Kavazanjian, E., Jr. (1984), "Probabilistic Seismic Stability of a Cohesionless Slope of Limited Extent," Proc. *8th World Conference on Earthquake Engineering*, Vol. III, San Francisco, California.

Kavazanjian, E. Jr., and Poepsel, P.H. (1984), "Elasto-Plastic Finite Element Analysis of the I-95 and Atchafalaya Embankment Foundations," *Geotechnical Engineering Research Report GT-2*, Department of Civil Engineering, Stanford University.

Kavazanjian, E. Jr. and Poepsel, P. (1984), "Numerical Analysis of Two Embankment Foundations," *American Society of Civil Engineers Special Technical Publication*, Sedimentation/Consolidation Models: Prediction and Validation.

Roth, R.A. and Kavazanjian, E. Jr., (1984), "Liquefaction Susceptibility Mapping for San Francisco," California, *Bulletin of the Association of Engineering Geologists*, Vol. XXI, No. 4.

Kavazanjian, E. Jr., Roth, R.A., and Echezuria, H. (1985), "Probabilistic Evaluation of Liquefaction Potential for San Francisco, California," *Journal of Geotechnical Engineering*, American Society of Civil Engineers, Vol. 111, No. GT1.

McCann, M.S., Jr., Franzini, J.B., Kavazanjian, E. Jr., and Shah, H.C. (1985), "Preliminary Safety Evaluation of Existing Dams," Vol. I and II, *John A. Blume Earthquake Engineering Center Technical Reports No. 69 and 70*, Stanford University.

Wang, J.N., and Kavazanjian, E., Jr. (1985), "Pore Pressure Development During Non-Uniform Cyclic Triaxial Loading," *John A. Blume Earthquake Engineering Center Technical Report No. 73*, Stanford University.

Hsieh, H.S., and Kavazanjian, E., Jr. (1985), "Automated Triaxial Cell for Measuring the At-Rest Earth Pressure Coefficient," *Geotechnical Engineering Research Report GT-3*, Department of Civil Engineering, Stanford University.

Hadj-Hamou, T.A., and Kavazanjian, E., Jr. (1985), "Seismic Stability of Gentle Infinite Slopes," *Journal of the Geotechnical Engineering*, American Society of Civil Engineers, Vol. 111, No. GT6.

Kavazanjian, E. Jr., Echezuria, H., and McCann, M.W. (1985), "RMS Acceleration Hazard for San Francisco," *Soil Dynamics and Earthquake Engineering*, Vol. 4, No. 3.

Kavazanjian, E., Jr., Borja, R.I., and Jong, H-L (1985), "Numerical Analysis of Time-Dependent Deformations in Clay Soils," Proc. *11th International Conference on Soil Mechanics and Foundation Engineering*, Vol. 2, San Francisco, California.

Borja, R.I., and Kavazanjian, E., Jr. (1985), "A Constitutive Model for the Stress-Strain-Time Behavior of 'Wet' Clay," *Geotechnique*, Vol. 35, No. 3.

Ho, C.L., and Kavazanjian, E., Jr. (1986), "Probabilistic Study of SPT Liquefaction Analysis," Proc. *In Situ '86*, ASCE Specialty Conference on Use of In-Situ Tests in Geotechnical Engineering, Blacksburg, Virginia, ASCE Geotechnical Special Publication No. 6.

Hsieh, H-S, and Kavazanjian, E., Jr. (1987), "A Non-Associative Cam-Clay Plasticity Model for the Stress-Strain-Time Behavior of Soft Clays," *Geotechnical Engineering Research Report No. GT-4*, Department of Civil Engineering, Stanford University.

Wang, J-N, and Kavazanjian, E., Jr. (1987), "A Non-stationary Probabilistic Model for Pore Pressure Development and Site Response Due to Seismic Excitation," *John A. Blume Earthquake Engineering Center Technical Report No. 84*, Stanford University.

Ho, C.L., Sarmiento, J.S., and Kavazanjian, E., Jr. (1988), "Stabilization of Liquefiable Samples During Transport," *Geotechnical Testing Journal*, ASTM, Vol. II, No. 1.

Wang, J-N, and Kavazanjian, E., Jr. (1988), "Non-Stationary Seismic Site Response with Pore Pressure," *Proc. Joint ST/EMD/GT Conference on Probabilistic Methods*, ASCE, Blacksburg, Virginia.

Kavazanjian, E., Jr., and Wang, J-N (1988), "Frequency Domain Site Response Analysis with Pore Pressure Softening," Proc. *9th World Conference on Earthquake Engineering*, Tokyo.

Kavazanjian, E. Jr., and Hsieh, H-S (1988), "A Creep-Inclusive Non-Associative Cam-Clay Plasticity Model," *Rheology and Soil Mechanics*, Elsevier.

Wang, J-N, and Kavazanjian, E., Jr. (1989), "Pore Pressure Development During Non-Uniform Cyclic Loading," *Soils and Foundation*, JSSMFE, Vol. 29. No. 2.

Ho, C.L., and Kavazanjian, E., Jr. (1990), "Reduction Factor for Liquefaction Potential Analysis," *Soil Dynamics and Earthquake Engineering*, Vol. 9, No. 6.

Hsieh, H-S, Kavazanjian, E., Jr., and Borja, R.I. (1990), "Double Yield Surface Cam-Clay Plasticity Model," *Journal of Geotechnical Engineering*, ASCE, Vol. 116, No. GT-9, pp. 1381-1401.

Borja, R.I., Hsieh, H-S., and Kavazanjian, E., Jr. (1990), "Double Yield Surface Model. II: Implementation and Verification," *Journal of Geotechnical Engineers*, ASCE, Vol. 116, No. GT-9, pp. 1402-1421.

Kavazanjian, E., Jr., Hushmand, B., and Martin, G.R. (1991), "Frictional Base Isolation Using a Layered Soil-Synthetic Liner System," Proc. *3rd U.S. Conference on Lifeline Earthquake Engineering*, ASCE Technical Council on Lifeline Earthquake Engineering Monograph No. 4, pp. 1139-1151.

Salah-mars, S. and Kavazanjian, E., Jr. (1992), "A Virtual Surface Concept for Nested Yield Surface Plasticity," *International Journal of Numerical Methods in Geomechanics,* Vol. 16, pp. 779-796.

Kavazanjian, E., Jr. (1993), "SASW Testing at Solid Waste Landfill Facilities," Proc. *National Science Foundation Workshop on Seismic Design of Solid Waste Landfills*, University of Southern California.

Derian, L., Gharios, K.M., Kavazanjian, E., Jr., and Snow, M.S. (1993), "Geosynthetics Conquer the Landfill Law," *Civil Engineering*, ASCE, Vol. 63, No. 12.

Snow, M.S., Bonaparte, R., and Kavazanjian, E. Jr. (1994), "Geosynthetic Composite Liner System for Subtitle D," Proc. *Waste Tech '94 Landfill Technology Conference*, National Solid Waste Management Association, Charleston, South Carolina.

Jacob, A., Thevanayagam, S., and Kavazanjian, E. (1994), "Vacuum-Assisted Consolidation of A Hydraulic Landfill," Proc. *Settlement '94*, American Society of Civil

Engineers, Geotechnical Special Publication No. 40, College Station, Texas, pp. 1249-1261.

Snow, M.S., Kavazanjian, E., Jr., and Sanglerat, T.R. (1994), "Geosynthetic Composite Liner System for Steep Landfill Side Slopes," Proc. *5th International Conference on Geotextiles, Geomembranes, and Related Products*, Singapore, pp. 969-972.

Kavazanjian, E., Jr., Snow, M.S., Matasovic, N., Poran, C. and Satoh, T. (1994), "Non-Intrusive Rayleigh Wave Investigations at Solid Waste Landfills," Proc. *1st International Congress on Environmental Geotechnics*, Edmonton, Alberta, pp. 707-712.

Kavazanjian, E., Jr. (1994), "Performance of a Geosynthetic Landfill Liner System in the 1994 Northridge Earthquake," An Interview, *Geotechnical Fabrics Report.*

Kavazanjian, E., Jr. and Matasovic, N. (1995), "Seismic Analysis of Solid Waste Landfills," In: *Geoenvironment 2000,* ASCE Geotechnical Special Publication No. 46, Vol. 2, pp. 1066-1080.

Kavazanjian, E., Jr., Matasovic, N. Bonaparte, R. and Schmertmann, G.R. (1995), "Evaluation of MSW Properties for Seismic Analysis," In: *Geoenvironment 2000,* ASCE Geotechnical Special Publication No. 46, Vol. 2, pp. 1126-1141.

Richardson, G.N., Kavazanjian, E., Jr. and Matasovic, N. (1995), "RCRA Subtitle D (258) Seismic Design Guidance for Municipal Solid Waste Landfill Facilities," EPA/600/R-95/051, United States Environmental Protection Agency, Cincinnati, Ohio, 143 p.

Augello, A.J., Bray, J.D., Kavazanjian, E., Jr., Matasovic, N., Herzig, R., and Seed, R.B. (1995), "Performance of the Solid Waste Landfills in the 1994 Northridge Earthquake," Proc. *International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics*, University of Missouri-Rolla, Rolla, Missouri, Vol. III, pp. 1395-1401.

Anderson, D.G., and Kavazanjian, E., Jr. (1995), Performance of Landfills Under Seismic Loading, Proc. *International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics*, University of Missouri-Rolla, Rolla, Missouri, Vol. III, pp. 1557-1587.

Matasovic, N., Kavazanjian, E., J., Augello, A.J., Bray, J.D. and Seed, R.B. (1995), "Solid Waste Landfill Damage Caused by 17 January 1994 Northridge Earthquake," In: Woods, Mary C. and Seiple, Ray W., Eds., *The Northridge, California, Earthquake of 17 January 1994: California Department of Conservation, Division of Mines and Geology Special Publication 116*, Sacramento, California, pp. 221-229.

Kavazanjian, E. Jr., Bonaparte, R., Johnson, G.W., Martin, G.R. and Matasovic, N. (1995), "Hazard Analysis for a Large Regional Landfill," In: *Earthquake Design and Performance of Solid Waste Landfills,* ASCE Geotechnical Special Publication No. 54, pp. 119-141.

Augello, A.J., Matasovic, N. Bray, J.D., Kavazanjian, E., Jr., and Seed, R.B. (1995), "Evaluation of Solid Waste Landfill Performance During the Northridge Earthquake," In: *Earthquake Design and Performance of Solid Waste Landfills,* ASCE Geotechnical Special Publication No. 54, pp. 17-50.

Castelli, R.J., Rice, P.M., and Kavazanjian, E., Jr. (1995), "Rehabilitation of Trout Run Dam," Proc. *Annual Meeting of the Association of State Dam Safety Officials*, Atlanta, Georgia, 17 September, pp. 603-612.

Matasovic, N., Kavazanjian, E., Jr. and Abourjeily, F. (1995), "Dynamic Properties of Solid Waste from Field Observations," Proc. *1st International Conference on Earthquake Geotechnical Engineering*, Tokyo, Japan, Vol. 1, pp. 549-554.

Yan, L., Matasovic, N. and Kavazanjian, E., Jr. (1996), "Seismic Response of Rigid Block on Inclined Plane to Vertical and Horizontal Ground Motions Acting Simultaneously," Proc. *11th ASCE Engineering Mechanics Conference*, Fort Lauderdale, Florida, Vol. 2, pp. 1110-1113.

Matasovic, N. and Kavazanjian, E., Jr. (1996), "Observations of the Performance of Solid Waste Landfills During Earthquakes," Proc. *11th World Conference on Earthquake Engineering*, Acapulco, Mexico, Elsevier Science Ltd. Paper No. 341, 8 p. (on CD ROM)

Kavazanjian, E., Jr., Matasovic, N., Stokoe, K.H., II, and Bray, J.D. (1996), "In-Situ Shear Wave Velocity of Solid Waste from Surface Wave Measurements," Proc. *2nd International Congress Environmental Geotechnics*, Osaka, Japan, Balkema, Vol. 1, pp. 97-104.

Kavazanjian, E., Jr., and Tanaka, M. (1997), "Geotechnics of Waste Landfill," General report, Proc. *2nd International Congress Environmental Geotechnics*, Osaka, Japan, Balkema, Vol. 3, pp. 1565-1569

Matasovic, N., Kavazanjian, E. Jr. and Yan., L. (1997), "Newmark Deformation Analysis with Degrading Yield Acceleration," Proc. *Geosynthetic '97*, Long Beach, California, Vol. 2, pp. 989-1000.

Kavazanjian, E., Jr., Matasovic, N. Hadj-Hamou, T., and Sabatini, P.J. (1997), "Design Guidance: Geotechnical Earthquake Engineering for Highways," *Geotechnical Engineering Circular No. 3*, DTFH61-94-C-00099, U.S. Department of Transportation, Federal Highway Administration, Washington, D.C.

Matasovic, N. and Kavazanjian, E., Jr. (1998), "Cyclic Characterization of OII Landfill Solid Waste," *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 124, No. 3, pp. 197-210

Matasovic, N. Kavazanjian, E., Jr., and Anderson, R.L. (1998), "Performance of Solid Waste Landfills in Earthquakes," *Earthquake Spectra*, EERI, Vol. 14, No. 2, pp. 319-334.

Matasovic, N., Kavazanjian, E., Jr., and Giroud, J.P., (1998) "Newmark Seismic Deformation Analysis for Geosynthetic Covers," *Geosynthetics International,* International Geosynthetics Society, Vol. 5, Nos. 1 - 2, pp. 237-264.

Kavazanjian, E., Jr. (1998), "Current Issues in Seismic Design of Geosynthetic Cover Systems," Proc. *Sixth International Conference on Geosynthetics,* Atlanta, Georgia, Vol. I, pp. 219-226.

Zornberg, J.G. and Kavazanjian, E., Jr. (1998), "Evaluation of a Geogrid Reinforced Slope Subjected to Differential Settlements," Proc. *Sixth International Conference on Geosynthetics,* Atlanta, Georgia, Vol. I, pp. 465-468.

Hendricker, A.T., Fredianelli, K.H., Kavazanjian, E., Jr., and McKelvey, J.A., III (1998), "Reinforcement Requirements at a Hazardous Waste Site," Proc. *Sixth International Conference on Geosynthetics,* Atlanta, Georgia, Vol. I, pp. 465-468.

Snow, M.S., Mansour, R., Swan, R.H., Jr., and Kavazanjian, E., Jr. (1998), "Variability of Interface Shear Strengths," Proc. *Sixth International Conference on Geosynthetics,* Atlanta, Georgia, Vol. I, pp. 439-442.

Kavazanjian, E., Jr., Matasovic, N. and Caldwell, J. (1998), "Damage Criteria for Solid Waste Landfills," Proc. *Sixth U.S. National Conference on Earthquake Engineering* (on CD ROM), Seattle, Washington, 31 May – 4 June.

Kavazanjian, E., Jr. (1999), "Proc. *TRB Workshop on New Approaches to Liquefaction Analysis,*" Federal Highway Administration Publication No. FHWA-RD-99-165, January, (CD ROM)

Kavazanjian, E., Jr. (1999) "Seismic Design of Bridge Foundations," Proc. *Mid-America Highway Seismic Conference,"* St. Louis, Missouri, 28 February - 3 March.

Kavazanjian, E. Jr. (1999) "Seismic Design of Solid Waste Containment Facilities," Proc. *Eighth Canadian Conference on Earthquake Engineering,* Vancouver, British Columbia, 13-16 June, pp. 51-68.

Kavazanjian, E., Jr., Matasovic, N. and Bachus, R.C. (1999), "Large-Diameter Static and Cyclic Laboratory Testing of Municipal Solid Waste," Proc. *Sardinia '99 - 7th International Waste Management and Landfill Symposium*, Cagliari, Italy, October, Vol. III, pp. 437-444.

Kavazanjian, E. Jr. (2000) "Geoenvironmental Performance Monitoring," Proc. Geo=Institute Conference on *Performance Confirmation of Constructed Geotechnical Facilities,* Geotechnical Special Publication No. 94, ASCE, April, pp. 21-40.

Evans, T.M., Meyers, D.K., Gharios, K.M., Hadj-Hamou, T., and Kavazanjian, E., Jr. (2000) "The Use of a Capillary Barrier Final Cover for Reclamation of a Closed Municipal Solid Waste Landfill," Proc. *3rd Annual Arid Climate Symposium*, Solid Waste Association of North America, New Mexico Chapter, Albuquerque, New Mexico, 12-14 April.

Bouazza, A. and Kavazanjian, E., Jr. (2000), "Characterization of Municipal Solid Waste Sites Using the Continuous Surface Wave Method," Proc. *GeoEng2000* (on CD ROM), Melbourne, Australia, 19-24 November.

Kavazanjian, E., Jr. (2001), "Design and Performance of Evapotranspirative Cover Systems For Arid Region Landfills," Proceedings, *36th Annual Engineering Geology and Geotechnical Engineering Symposium*, 28-30 March 2001, University of Nevada, Las Vegas, Nevada, pp. 11-26

Kavazanjian, E. Jr., and Matasovic, N. (2001) "Seismic Design of Mixed and Hazardous Waste Landfills," State of the Art Paper No. 11, Proceedings, *Fourth International Conference on Recent Advances in Geotechnical Earthquake Engineering,* University of Missouri, Rolla, 27-31 March, San Diego, California, on CD ROM

Kavazanjian, E. Jr., Hendron, D. and Corcoran, G.T. (2001) "Strength and Stability of Bioreactor Landfills," Proceedings, *6th Annual Landfill Symposium*, Solid Waste Association of North America, Silver Springs, Maryland, 18-20 June, San Diego, pp. 63-72

Kavazanjian, E., Jr. (2001) "Mechanical Properties of Municipal Solid Waste," Proc. *Sardinia '01 - 8th International Waste Management and Landfill Symposium*, Cagliari, Italy, October, Vol. III, pp. 415-424

Menq, F-Y, Stokoe, K.H., II, and Kavazanjian, E., Jr. (2001) "Dynamic Properties of Municipal Waste from Large-Scale Resonant Testing," Proc. *Sardinia '01 - 8th International Waste Management and Landfill Symposium*, Cagliari, Italy, October, Vol. 111, pp. 435-444.

Bouazza, M., and Kavazanjian, E. Jr. (2001) "Construction on Former Landfills," Proc. *2nd ANZ Conference on Environmental Geotechnics,* Newcastle, Australia, 28-30 November 2001

Zornberg, J.G., and Kavazanjian, E. (2001). "Prediction of the Performance of a Geogrid-Reinforced Slope Founded on Solid Waste." *Soils and Foundations*, Vol. 41, No. 6, December.

B

Geotechnical Summary Report
Third Runway Embankment and
MSE Retaining Walls
Seattle-Tacoma International Airport

HARTCROWSER
Delivering smarter solutions

Prepared for
The Port of Seattle
for Presentation to
The U.S. Army Corps of Engineers

November 2, 2001
4978-06

AR 014185

HARTCROWSER
delivering smarter solutions

www.hartcrowser.com

Anchorage
Boston
Chicago
Denver
Fairbanks
Jersey City
Juneau
Long Beach
Portland
Seattle

Geotechnical Summary Report
Third Runway Embankment and
MSE Retaining Walls
Seattle-Tacoma International Airport

Prepared for
The Port of Seattle
for Presentation to
The U.S. Army Corps of Engineers

November 2, 2001
4978-06

Prepared by
Hart Crowser, Inc.

MICHAEL J. BAILEY
STATE OF WASHINGTON
22484
REGISTERED
PROFESSIONAL ENGINEER
EXPIRES 12/13/01

11/2/01

Michael Bailey, P.E.
Project Manager

1910 Fairview Avenue East
Seattle, Washington 98102-3699
Fax 206.328.5581
Tel 206.324.9530

GEOTECHNICAL SUMMARY REPORT
THIRD RUNWAY EMBANKMENT AND MSE RETAINING WALLS
SEATTLE-TACOMA INTERNATIONAL AIRPORT

EXECUTIVE SUMMARY

This report describes the engineering process used to address design issues related to soil conditions, groundwater, and potential earthquakes for the proposed Third Runway at Seattle-Tacoma International Airport (STIA). Overall, the runway project will include placement of 17,000,000 cubic yards of compacted fill, 3,000,000 cubic yards of excavation, and construction of three "mechanically stabilized earth" (MSE) retaining walls that range from 50 to 135 feet in maximum height.

The executive summary of this report describes its purpose, general contents of the report, and results of the engineering analysis. A key part of the work described herein has been the involvement of an independent technical review board composed of distinguished experts to provide input into the geotechnical design process.

The main part of this report summarizes the geotechnical data collection and engineering analyses accomplished over a multi-year period by the Port of Seattle. The Seattle District, US Army Corps of Engineers (Corps) requested this executive summary as part of its review of the Third Runway Project.

Scope and Purpose of This Report

The scope of this report is to address the following:

- Introduce the reader to the design team and explain what each firm's role has been, including the involvement of outside reviewers;
- Describe the main features of the embankment and MSE retaining walls that are addressed in this report;
- Summarize information that has been collected on soil and groundwater conditions at the Third Runway site;
- Generally describe how the Port has studied the risk posed by earthquakes, and how seismic hazards are being addressed in the design process;
- Discuss the methods of engineering analyses used for design of the embankment slopes and retaining walls; and
- Describe how construction will include specific measures to mitigate problematic soil conditions, assure stability and meet seismic performance criteria.

The purpose of this report is to provide the Corps with a summary of the geotechnical work that has been accomplished for the Third Runway project, including references to other reports prepared by the Port's design team that provide more comprehensive discussion and details.

"Road Map" for Readers

A detailed table of contents, with lists of figures and tables, follows this executive summary. Thereafter:

- Section 1 is a general introduction to the Third Runway project and the engineering design team.
- Section 2 describes the geotechnical design process.
- Section 3 explains how soil and groundwater information was obtained and provides a geologic description of the project site.
- Section 4 discusses the methods of geotechnical engineering analyses used.
- Section 5 describes how the MSE wall design has incorporated geotechnical input and the results of independent checks and review.
- Section 6 discusses how construction will include "subgrade improvements" to mitigate problem soil conditions, and assure stability.

A bibliography of other reports that present geotechnical information for the Third Runway project follows the main text, along with a list of other technical references. Tables, figures, and the oversize plates cited in the text are included at the end of the report.

Engineering Quality Assurance

The Port of Seattle has assembled a team of notable engineering firms (HNTB, Hart Crowser, and RECo) to the design the Third Runway embankment and retaining walls. Qualifications of these firms to fill their specific roles, along with other experts who are providing support to the design team are discussed as part of the introduction to the design process, later in this report.

MSE retaining walls for the Third Runway are being designed in accordance with, and exceeding criteria established by the American Association of State Highway and Transportation Officials (AASHTO). Design of the project features is being accomplished with methods that are well-established and widely accepted by the engineering community. In addition, the Port has utilized advanced engineering analysis to check the design and evaluate performance of the Third Runway embankment and retaining walls. The Port's design meets or exceeds comparable "factor of safety" criteria used by the Corps for design of earth embankments (levees) and retaining walls.

To support the design team, the Port has used outside technical reviewers to provide independent assessment of various parts of the design process. The Embankment Technical Review Board (ETRB) members include Dr. James K. Mitchell, P.E., an expert in soil behavior, ground improvement, and earth reinforcement; Dr. I.M. Idriss, P.E., a recognized authority on earthquake engineering; and Dr. Barry Christopher, P.E., an internationally recognized expert in MSE wall design, construction, and performance.

The ETRB has worked closely with the Port's design team to develop an understanding of the Third Runway project and subsurface conditions at the site. The Board has provided detailed recommendations for improving design analyses and implementation of additional test and sophisticated analyses to improve the design. The Port's design team has addressed the Board's recommendations, and thereby enhanced the design. In addition to the ETRB, the Port has utilized other experts to provide independent technical input to the Third Runway design team, in several other specific instances since 1998.

This report describes specific input from the ETRB and others at different parts of the design process, which provides assurance that the work accomplished meets the highest technical standards.

Seismic Performance Goals for the Embankment and Walls

The Port has adopted seismic performance goals for the Third Runway embankment and MSE walls. The purpose of these goals is to clearly state the result of the geotechnical design process in terms that are easier to understand compared to the numeric factors of safety specified by the AASHTO code.

The Port of Seattle's design team gave considerable attention to selecting the level of earthquake shaking that would be used as the basis for design. This process considered statistical extrapolation of seismic data for our region, and explicitly considered the effect of variations in size, location and attenuation of future earthquakes. The methods used were subjected to scrutiny by the design team and the ETRB experts, and analyses by well-established methods were checked by independent methods to verify appropriateness of the design.

The Third Runway project is being designed as a "structure of ordinary importance" similar to large public buildings and other transportation infrastructure such as bridges and highways. In technical terms, the project is being designed to perform well for seismic ground motions that have a 10 percent probability of being exceeded in 50 years - or in other words, the level of shaking that has an average return period of 475 years.

AR 014189

Specific performance goals for the Third Runway project are to meet the following conditions for this design level of shaking:

- The MSE walls and embankment fill will remain stable. Some deformation is acceptable (up to a few feet) provided stress in the retaining wall materials are typically below the value allowed by the AASHTO code;
- There will be no wetland or creek impacts due to seismic shaking of the embankment or MSE walls; and
- There will be no operational impacts to the new runway related to movement of the embankment slopes and walls during an earthquake.

The engineering analyses described in this report have been accomplished iteratively with design modifications to assure the completed embankment slopes and MSE retaining walls will meet the performance objectives. As needed, the design has been modified by increasing the extent of "improvement" of subgrade soils and/or by increasing length or embedment of the MSE reinforcing. In addition to using the conventional engineering analyses specified by AASHTO, the Port has utilized advanced methods of analysis that are more typically used for design of dams impounding reservoirs.

The remainder of this report provides additional technical detail to expand on information provided in this executive summary.

CONTENTS

Page

AR 014191

CONTENTS (Continued)

TABLES

FIGURES

PLATES

GEOTECHNICAL SUMMARY REPORT
THIRD RUNWAY EMBANKMENT AND MSE RETAINING WALLS
SEATTLE-TACOMA INTERNATIONAL AIRPORT

1.0 INTRODUCTION

This report provides a summary of the process used for geotechnical site investigations, laboratory testing, and analyses used for design and construction of the Third Runway embankment and MSE walls at Seattle-Tacoma International Airport (STIA).

Since 1998, the Port of Seattle has obtained detailed information on soil and groundwater conditions at the site of the proposed Third Runway. This information has been incorporated into the design so that construction will be appropriate for site conditions and conform to applicable building codes and engineering standards. A significant part of this process is to identify seismic hazards and assure that the completed facility meets the seismic performance goals set by the Port.

Geotechnical explorations and tests to identify and measure subsurface soil and groundwater conditions have been accomplished in phases, with intermediate analyses used to evaluate potential stability of the embankment and MSE walls and to identify areas where additional data collection was needed. Methods and results have been extensively reviewed and modified as needed to assure the completed project is safe and will perform as designed.

In several instances, the design approach utilized by the Port significantly exceeds the normal standard of care for transportation infrastructure, and incorporates techniques that are more commonly used for earthen dams. Clearly, performance of the Third Runway project is not as critical as a dam would be from the perspective of safeguarding human life. However, the Port of Seattle recognizes the project is a significant engineering structure, and the Port has utilized sophisticated engineering methods in recognition of the project location adjacent to sensitive and valued surface water resources, and the local community.

The purpose of this geotechnical summary report is to provide the US Army Corps of Engineers (Corps) with documentation of the geotechnical design process that has occurred, and the work in progress, which will lead to completion of design for the embankment and MSE walls.

AR 014193

1.1 Project Overview

The proposed Third Runway will be constructed in part on an embankment of compacted earth fill, so that the new runway elevation matches the existing airfield. Part of the runway will also be located on native soils near the south end of the existing airfield.

To accommodate the slope of the existing terrain, the new embankment will vary up to a maximum fill thickness of about 165 feet. The new embankment is being constructed as a zoned earth fill, with specific types of soil materials and compaction requirements used in different areas to provide necessary stability, drainage and settlement characteristics. Overall, the new embankment will include about 17,000,000 cubic yards of compacted earth fill. Approximately 3,000,000 cubic yards will be excavated onsite, leaving 14,000,000 cubic yards of fill to be imported.

The new embankment will be constructed on the west side of the existing airfield, see Figure 1. New embankment side slopes will have an average inclination of 2H:1V. Three retaining walls will be used to limit the extent of embankment slope from impacting sensitive portions of Miller Creek and adjacent tributary wetlands. These walls will have exposed faces that range up to maximum heights of 50 to 135 feet above ground.

The proposed retaining walls will be constructed of "mechanically stabilized earth" using engineering techniques more than 30 years old that use steel or other material to reinforce soil (FHWA 2001). The Port of Seattle evaluated eight types of retaining wall, and more than 60 wall and slope geometric arrangements before selecting the proposed MSE walls for the project. The methods and results of that evaluation are presented in the report entitled: *Draft Evaluation of Retaining Wall/Slope Alternatives to Reduce Impacts to Miller Creek Embankment Station 174+00 to 186+00, Third Dependent Runway,* that was prepared for the Port by HNTB Corporation, Hart Crowser, Inc., and Parametrix in April 1999. Note that the documents cited herein are listed in the bibliography at the end of this report (e.g., see HNTB, Hart Crowser, and Parametrix 1999).

The specific type of MSE walls being designed for the Third Runway utilize strips of steel layered in the compacted soil fill, and a relatively thin reinforced concrete facing to form a near vertical retaining wall face. MSE walls have been used around the world, with exposed face heights of up to 140 feet. This type of wall provides the advantages of very good seismic performance along with being very cost-effective. The completed walls will not impede groundwater seepage, or reduce base flow to the wetlands and Miller Creek, as discussed later in this report. (Note that a companion summary prepared for the Corps, provides additional detail on the hydrogeologic analyses of the Third Runway and adjacent wetlands and creeks; see Hart Crowser 2001l).

1.2 Embankment and MSE Wall Design Team

The Port of Seattle design team for the Third Runway embankment and MSE walls includes internationally recognized engineering firms and a distinguished independent review board. Figure 2 presents an organization chart for the project.

HNTB Corporation is the engineering project manager and civil engineer for the Third Runway project. In business since 1914, HNTB provides engineering and architectural design, planning and construction management for major transportation infrastructure projects. Recent airport experience includes major airport expansion and renovation projects at George Bush Intercontinental Airport in Houston, Midway Airport in Chicago, and Dulles international Airport near Washington DC.

HNTB has selected the Reinforced Earth Company (RECo) to design the MSE walls for the Third Runway project, and Hart Crowser Inc. to provide geotechnical engineering services.

- RECo was chosen as MSE wall designer for the Port of Seattle since they have more extensive experience with design and construction of high MSE walls than anyone else in the world. RECo has designed and successfully constructed more than twenty thousand MSE walls (FHWA 2001), including 12 that are more than 90 feet high, and have been successfully constructed. RECo designed two MSE walls that were built to about the same height as the maximum proposed wall height at SeaTac: a 137-foot-high wall built in 1979 in South Africa and a 133-foot-high wall built in Hong Kong in 1993. These walls were successfully constructed and have preformed well for some time.

- Hart Crowser Inc. is a local geotechnical engineering firm with more than 25 years experience in the Seattle area. Hart Crowser has been lead geotechnical engineer on major infrastructure projects such as the US Navy Home Port in Everett, WA and high-rise buildings in downtown Seattle, such as the Millennium Tower. Hart Crowser has been responsible for stability analyses for the right abutment at Mud Mountain Dam for the Corps of Engineers, Cedar Embankment at Chester Morse Lake for the Seattle Water Department, as well as major tailings embankments for the mining industry.

Hart Crowser has been responsible for design of MSE reinforced slopes that have been successfully constructed up to 150 feet in height.

Hart Crowser has retained expert subconsultants from the University of Washington and elsewhere to provide special geotechnical assistance on the Third Runway design team. These experts include Professor Robert Holtz, PhD, P.E., an internationally recognized MSE expert; and Professor Steve Kramer, PhD, P.E., an expert in earthquake engineering. Other expert subconsultants utilized for the Third Runway Project including Professor Pedro Arduino, University of Washington, for assistance in computer modeling; and Dr. John Hughes who is a specialist in *in situ* testing using the soil pressure meter. Specialty testing firms were also used to assist in geophysics (GeoRecon International); cone penetrometer testing (Northwest Cone); and drilling for soil sampling and installation of monitoring wells (Holt Drilling).

1.3 Embankment Technical Review Board (ETRB)

HNTB has retained the services of an internationally recognized group of eminent engineers to form a special technical review board, to provide independent technical review for the Third Runway project. Detailed resumes for the board members have been submitted to the Corps as part of the record for the 404 permit process. The board members include:

Dr. James K. Mitchell, P.E., is a University Distinguished Professor Emeritus at the Virginia Polytechnic Institute and State University and former Chairman of the Civil Engineering Department at the University of California, Berkeley. Professor Mitchell is an expert in soil behavior, ground improvement, and earth reinforcement.

Dr. I.M. Idriss, P.E., is Professor of Civil Engineering at the University of California at Davis. Professor Idriss is a recognized authority on earthquake engineering and on seismic performance of embankments and other soil structures.

Dr. Barry Christopher, P.E., is an independent geotechnical engineering consultant and internationally recognized expert in MSE wall design, construction, and performance.

The Port's Technical Review Board is coordinated by **Mr. Peter Douglass, P.E.** Mr. Douglass is an independent geotechnical consultant who has earned advanced degrees in civil engineering and geology. Mr. Douglass has more than 30 years of geotechnical engineering experience in the Seattle area as well as around the world.

The ETRB has been given the engineering data, design reports, results of calculations, and MSE design plans to date, for review and comment. Some or all of the members of the Board met with the Port's design team six times in the period November 2000 to October 2001, and have participated in several conference calls to provide expert input to the ongoing site explorations, analyses and design.

Working closely with the Port's design team, the ETRB has developed a good understanding of geotechnical issues pertinent to design and construction of the Third Runway. Drawing on their extensive expertise with analysis of earthquakes, soil reinforcement, and soil behavior, the Board has provided recommendations for improving the accuracy of analyses by the design team and use of sophisticated engineering methods to confirm results. Equally important is the practical knowledge and understanding the ETRB has from their extensive experience in construction and performance evaluations of large embankments and MSE walls around the world.

1.4 Other Independent Review Consultants

During preliminary stages of design, the Port of Seattle reviewed eight different types of retaining wall and more than 60 wall/slope combinations before selecting the proposed MSE wall configuration (HNTB, Hart Crowser, and Parametrix 1999). The evaluation of alternatives by the Port's design team was independently reviewed by qualified geotechnical engineers at Shannon & Wilson Inc. Shannon & Wilson is a highly regarded local engineering firm that is **not** part of the Port's Third Runway design team.

Shannon & Wilson concluded that the proposed MSE retaining walls are "most appropriate" for this site. Their findings were documented by letter and submitted to the Corps of Engineers as part of the public record for the Section 404 permit process.

The Port also obtained technical assistance in developing the scope for MSE wall design from Mr. Tony Allen, P.E. Mr. Allen is the State Geotechnical Engineer for the Washington State Department of Transportation (WSDOT). He has participated extensively in developing national standards for MSE design through his work with the American Association of State highway and Transportation Engineers (AASHTO).

AASHTO has developed a rigorous code for design of MSE walls based on the experience of numerous state transportation agencies, other engineering organizations, and research by the Federal Highway Administration (FHWA). This code is part of AASHTO's "Standard Specifications for Highway Bridges" and is the standard of the industry for design of MSE walls. The current version of is presented in the 16th edition, 1996, which has been updated with interim addenda through 2000 (AASHTO 1996-2000). Reference to the AASHTO code in this report indicates the provisions of the 1996 edition with inclusion of the interim addenda through 2000 (which is the most current addendum).

Based in part on recommendations from Tony Allen, the Port is designing the Third Runway MSE walls in accordance with the AASHTO code. Mr. Allen also recommended the Port utilize another industry standard, the HiTec Protocol, another industry standard as part of checking the MSE wall designs for the Third Runway project, and this is being done by HNTB.

2.0 GEOTECHNICAL SUMMARY

This section of the report provides a discussion of the geotechnical work completed and current progress of design of the Third Runway embankment and MSE walls that is discussed later in this report. Engineering aspects of the project that were described in a previous report to the Corps (Hart Crowser 1999c) are substantially unchanged.

This report summarizes the performance standards, and codes and standards that guide the geotechnical design process for the Third Runway project. This summary also describes the extensive soil explorations, tests and analyses that have been completed and/or are ongoing as part of final design. This report notes where additional geotechnical information is documented in the reports and technical memoranda that are listed at the end of this report, along with other references.

2.1 Performance Standards for Geotechnical Design

The geotechnical design for the Third Runway project conforms to several types of design performance standards. These include satisfaction of numerical requirements in the AASHTO code for design of MSE walls, as well as the readily understood seismic performance goals that were outlined in the executive summary to this report.

The Port has used a great deal of care to identify applicable design requirements and to verify that its design satisfies all the requirements of the AASHTO code. The Port has also addressed other engineering methods and criteria as a check on its design. In particular, the Port has accomplished deformation modeling with sophisticated computer modeling tools (programs referred to as QUAD4 and FLAC, that are described later in this report). Deformation models are important because they provide "real world" estimates of performance (such as "how far will a wall move during an earthquake?"). The deformation models used by the Port also provide a detailed picture of how stresses in the embankment and the MSE walls will change during earthquake shaking.

The approach used by the Port enables verification that not only does the design satisfy the code requirements, but also that estimated movements of the embankment and MSE walls are acceptable.

The Port has designed the Third Runway embankment and MSE walls to meet the following seismic performance requirements:

- MSE walls and fill will remain stable during and following the design level of earthquake shaking (average return interval of 475 years). Some deformations and/or cosmetic damage to the walls are acceptable provided the stresses are not large enough to cause failure.

- There will be no wetland or creek impacts from the embankment or MSE walls due to design level earthquake shaking. Movement will be limited to prevent soil sloughing or release of water that would impact surface water resources adjacent to the airfield.

- There will be no runway operational impacts due to the movement of the embankment slopes or MSE walls subject to the design level of earthquake shaking.

Note that the third performance criterion is specific to the embankment slopes and walls nearest to Miller Creek and adjacent wetlands. Potential effects of liquefaction on pavement within the interior part of the airfield have not been completed as part of the present study.

The design team is able to modify design of the subgrade improvements, MSE reinforcing, and/or the embankment materials and compare the estimated amounts of deformation for representative areas of the project, by the analyses detailed in this report. Seismic deformations analyzed to date for the final design configuration are typically well under a foot, and in some cases up to several feet, based on two independent types of analysis (FLAC and Newmark analyses, see Section 4.2 of this report). Rather than specify a single value for maximum allowable deformation, the design team is reviewing the results of the analyses to assess whether estimated deformations for different areas meet the performance criteria above. For comparison, allowable deformation of up to about three feet is commonly considered acceptable for slopes and earth embankments (ASCE 1983 and Seed 1979).

The Port's geotechnical design procedures and resultant Factor of Safety for each specific analysis meet all AASHTO criteria, and are consistent with procedures used by the Corps (EM 1110-2-2502; EM 1110-2-1913; and ER 1110-2-1806) for design of retaining walls and earth embankments for levees, (Corps 1989, 1995, and 2000). The Port's design significantly exceeds AASHTO requirements by including sophisticated deformation analyses and independent peer review input from the ETRB and others.

HNTB is using the "HiTec Protocol" as a guide for their independent check on RECo's design. The HiTec Protocol (CERF 1998) was developed by the Civil Engineering Research Foundation, an affiliate of the American Society of Civil Engineers, working in conjunction with FHWA and various state departments of transportation. Use of this protocol to check the design documents provides verification that the design includes all the elements found necessary for MSE walls to meet criteria developed by FHWA and the states.

2.3 Subsurface Explorations and Tests

Subsurface exploration and testing to determine soil and groundwater conditions affecting Third Runway design have been underway since the environmental review process for the project in the mid-1990s. The Port has used a phased approach to collect information for different parts of the site, with additional explorations accomplished as needed to better define conditions in particular areas. This report describes how 218 soil borings, 156 test pits, and other explorations have been used to identify and document soil and groundwater conditions; as the basis to assess environmental impacts and for design of the Third Runway.

Initially the subsurface exploration and test program accomplished by the Port of Seattle was based on local geotechnical experience and the results of initial observations. Existing mapped soils information was supplemented with soil borings and test pits to define baseline conditions for environmental review (FAA 1996 and 1997 and AGI 1996).

Additional explorations and tests were accomplished in specific areas to provide detailed information for related projects, conceptual design of the runway, and on-site borrow areas (CivilTech 1997, HWA Geosciences 1998, AGI 1998, and Hart Crowser 1998 and 1999a). A detailed description of the project was prepared for the Corps (Hart Crowser 1999c) with an accompanying subsurface conditions data report (Hart Crowser 1999b).

Subsurface information was subsequently obtained as part of a phased investigation that first addressed the locations for the three proposed MSE walls

Finally, it is notable that the Port's design team considered embankment and wall performance over a wide range of circumstances. For instance, the Port checked and verified that the MSE reinforcing stress and deformation levels would still be acceptable if the design level earthquake happened after the reinforcing strength was reduced by the calculated corrosion loss corresponding to a 100-year service life. This combination of the assumed long-term corrosion loss prior to occurrence of the design earthquake is an example of the Port's conservative approach to design.

2.2 Codes and Standards

Design of the Third Runway is covered by the Washington State regulations covering the practice of Professional Engineering (Chapter 18.43 RCW). The senior engineers supervising the work described in this report are Professional Engineers, licensed by the State of Washington, employed by experienced engineering firms such as Hart Crowser, HNTB, and RECo.

The Port's design team reviewed applicable engineering codes and standards, and decided to design and construct the Third Runway MSE walls in accordance with the current edition of the AASHTO code and its interim updates. (AASHTO 1996-2000) and by reference the FHWA standards on MSE walls (FHWA 1997). This decision was based on research contacts with other organizations and companies designing and/or involved with construction of MSE walls, including Professor Robert Holtz, University of Washington; Mr. Tony Allen, WSDOT; and Mr. James (Mickey) McGee, Georgia DOT).

In accomplishing our work, the Port's design team has also referred to other standards of practice for engineering works, such as the engineering manuals developed by the U.S. Army Corps of Engineers (EM 1110-2-2502, EM 1110-2-1913, and ER 1110-2-1806). Geotechnical design work for the Third Runway is similar to what the Corps would require for design of MSE walls and earth embankments (levees), as is also discussed later in this report.

Historically, safety of earth structures such as embankment slopes and retaining walls has been evaluated by stability analyses, using "factors of safety" to assess adequacy of the design relative to the loads expected during the lifetime of the structure. In its simplest form, a "factor of safety" is the ratio of the forces tending to maintain stability divided by the forces tending to cause instability. The AASHTO code (and other standards such as Corps documents EM 1110-2-2502, EM 1110-2-1913, and ER 1110-2-1806) specifies target factors of safety that the design must achieve for specific methods of analysis, and/or goals of analysis where alternative methods of analysis are determined by site-specific conditions.

(Hart Crowser 2000b (North or NSA Wall), 2000d (South Wall), and 2000f (West Wall)).

The type and frequency of subsequent explorations and testing were determined from assessment of the project's geologic environment; the extent of variation observed in initial test results; and additional data needs for specific parts of project design (Hart Crowser 2000j and 2001b and Appendix C of Hart Crowser 2001j). The design team had input from the ETRB in identifying the need for the final explorations and tests.

Field and laboratory work was accomplished in general accordance with standards developed by the American Society for Testing and Materials (see ASTM 2001 for current details). Table 1 summarizes the subsurface explorations that were accomplished; Table 2 lists the laboratory analyses that were used.

2.4 Seismic Basis of Design

The Port's design team made a considerable effort to select a reasonable basis of design to evaluate seismic effects on the Third Runway embankment and MSE walls. After review of procedures used for seismic design of other major structures and facilities, the Port of Seattle design team selected a probability-based approach that utilizes measurements from previous earthquakes throughout the Pacific-Northwest region, to predict the level of future seismic shaking at Sea-Tac (Hart Crowser 2000e and 2001a).

The design team completed a site-specific probabilistic seismic hazard assessment (PSHA) that utilizes current attenuation relationships and earthquake data, which have been peer-reviewed and are extensively used in Seattle and elsewhere for design of bridges and major buildings. The PSHA produced a relationship between the peak seismic acceleration and average recurrence period specific to the project site.

The Port of Seattle is basing design on the level of seismic shaking that has a 10 percent probability of exceedence in 50 years and an average return period of 475 years. Design using the 475-year seismic level of shaking is reasonable for the Third Runway facility. This level of event is commonly used for transportation facilities of normal importance, such as highway bridges and public buildings. While the Third Runway embankment and retaining walls are significant structures; they are not essential to airport operations. Potential damage to the Third Runway that might occur from an earthquake larger than the basis of design event would be similar to what might occur for other transportation facilities that use similar design standards. There is no risk of catastrophic loss of life due to seismic effects on the Third Runway, such as might result from failure of a dam or nuclear power plant.

Design for the level of shaking selected for the Third Runway is consistent with the approach that has been used for other major construction at STIA (e.g., the current South Terminal Expansion Project—a building that has thousands of people in it every day). The Third Runway design specifically addresses both the amount of movement that will occur as well as the stresses that will develop within the embankment and MSE walls as a result of earthquake shaking.

The design included development of several ground motions that were used in progressively more sophisticated analysis as design has proceeded. This aspect of design includes expert input from the University of Washington and has been closely scrutinized by the ETRB. Final design includes evaluation of stability and deformation for three ground motions (acceleration time history records) that were selected to represent the range of shaking obtained from the PSHA, as well as a ground motion from a deterministic source (the Seattle Fault) corresponding to a 475-year return period.

2.5 Stability and Deformation Analyses

The basic design approach for the Third Runway embankment and retaining walls is to use limit equilibrium stability analyses to determine the extent of subgrade improvement needed to meet minimum target factors of safety for different load conditions. For the MSE walls, the analyses included both global stability (to evaluate potential failure surfaces that extend behind and below the MSE reinforcing) as well as compound stability (to evaluate potential failure surfaces that pass through the reinforced soil zone). Reinforcement thickness, length, and/or embedment were increased as needed to meet target factors of safety. As a final check, deformation analyses are being used to verify the design will meet the Port's performance standards.

Limit equilibrium stability analyses were used to assess stability of the embankment including its MSE reinforced wall sections. Representative cross sections of the Third Runway embankment and retaining walls were analyzed for stability under the following load conditions:

- End of construction;
- Steady state;
- Seismic; and
- Post-liquefaction.

Cross sections were selected for analysis to represent the fill height, shape or geometry of the embankment/wall cross section, and the range in observed subsurface conditions. In most cases, our analyses showed that stability was more influenced by the strength of the existing subgrade soils, than the strength of the embankment or MSE fills, and "subgrade improvement" was needed to meet target factors of safety in specific areas (as described in Hart Crowser 2000 g). In some cases, increased length or depth of embedment of the MSE reinforcement was needed to meet target factor of safety (Hart Crowser 2000m, 2001g, and 2001k).

Two types of deformation analysis are being used to independently check performance of the Third Runway embankment and MSE walls.

- One method uses a finite difference program (FLAC) to calculate changes in stress and strain to simulate construction, and effects of the acceleration time history for seismic shaking. This analysis also considers the effect of reduced soil strength and stiffness due to liquefaction and cyclic loading.

- The other method uses a finite element program (QUAD4) to calculate accelerations throughout the embankment and MSE walls, and calculates displacements that occur when acceleration exceeds the yield acceleration for different parts of the embankment, using the Newmark method.

2.6 MSE Wall Design

MSE walls for the Third Runway are being designed to satisfy the following criteria:

1) Design requirements in the AASHTO code for MSE walls (AASHTO 1996-2000);

2) RECo in-house criteria, which include results of both theoretical and empirical methods of analysis, and performance criteria based on construction of similar walls;

3) Verification that RECo's design meets the target factor of safety criteria for both global and compound stability (as described above);

4) Verification that the proposed design will result in acceptable deformations for the design level of seismic shaking; and

5) Other functional and aesthetic requirements established by the Port.

All the analyses of the MSE sections were based on the calculated reinforcing section at the end of a 100-year performance period (i.e., including allowance for corrosion).

Design of the MSE walls is well along, including submittal of 30 percent draft plans, calculations, and quality assurance documents by RECo, and review by the rest of the design team (HNTB 2001).

2.7 Geotechnical Aspects of Construction

The culmination of the tests and analyses described in this report is the production of construction contract documents that show how the embankment and MSE walls must be constructed to achieve the design expectations. The limits of subgrade improvement, which were selected by design to meet target factor of safety in the stability analyses, will be shown on construction plans with accompanying Specifications that include detailed information on the quality of construction required.

Within the areas where subgrade improvements are needed, the Port plans to excavate the problematic soils (generally loose saturated sands, soft to stiff silt and clay soils, and peat) and replace them with densely compacted select fill. The Port evaluated nine alternative methods of subgrade improvement (Hart Crowser 2000g) and selected removal and replacement of problem soils (sometimes referred to as overexcavation and replacement) as the most desirable alternative because it will provide the highest level of ground improvement and the best quality control among the available alternatives.

The construction contract documents for the Third Runway project also specify the length, thickness, spacing, and arrangement of steel reinforcing strips that support the MSE walls, and the allowable soil types and compaction requirements needed to assure the constructed embankment meets the criteria used to achieve the target factors of safety and anticipated deformations.

The remainder of this report presents information on the soil and groundwater data used for design, the methods of geotechnical analyses that were used, and input of geotechnical input to the MSE design. Section 6.0 provides additional detail on geotechnical aspects of the proposed construction process.

3.0 SOIL AND GROUNDWATER DATA USED FOR DESIGN

This section of the report provides a summary of the methods of investigation used to assess subsurface conditions at the project site and an overview of geologic conditions that influence design. The final part of this section discusses selection of representative soil properties for use in the stability analyses.

3.1 Subsurface Explorations and Soil Tests

A large number of both conventional and special subsurface explorations have been accomplished to obtain geotechnical engineering parameters for the Third Runway project. These explorations are summarized in Table 1, and shown on a Site and Exploration Plan, Plates 1, 2 and 3, included at the back of this report.

Preliminary Explorations

As part of the environmental impact assessment and initial planning for the Third Runway project, the Port of Seattle accomplished 91 soil borings and a number of test pits and hand auger explorations (AGI 1996 and 1998). The borings were typically accomplished with hollow-stem auger or mud rotary drilling techniques, using the Standard Penetration Test (SPT, per ASTM D 1586) to collect soil samples and information on soil density or consistency. (Note throughout this report, applicable procedures developed by the American Society for Testing and Materials, are referred to simply by their test method designation. See ASTM 2001 for complete details). Nineteen of the initial borings were completed as groundwater observation wells.

Geotechnical Design Phase Explorations

During the geotechnical design phase, Hart Crowser completed an additional 127 hollow-stem auger borings, again using SPT to collect soil samples. At some of these boring locations, parallel borings were also drilled to obtain thin wall (Shelby) tube samples for laboratory testing. (These additional borings were not counted or numbered separately because they were merely to collect additional undisturbed soils samples at specific locations where the primary borings had been used to identify the soil strata).

Hart Crowser completed 65 of the design phase explorations as groundwater monitoring wells. All monitoring well locations were surveyed and groundwater level observations were recorded over a period of 1 to 3 years.

In addition to the borings, the main geotechnical design phase included 122 test pits excavated with a track-hoe, and numerous shallow hand auger explorations. Cone penetrometer test (CPT) soundings were completed at 48 locations to obtain information on stratigraphy, strength and stiffness of fine-grained soils (primarily silt and clay), as well as soil pore pressure parameters.

Additional Special Field Tests

During the design phase, a number of other special field tests were accomplished to better define subsurface conditions. These tests included:

- Two types of infiltration tests were used to evaluate effects of construction on groundwater, and stormwater infiltration. The tests included ring infiltrometer tests accomplished with a double-ring apparatus in test pits, and falling head infiltration tests accomplished in well casings;

- Vane shear tests were accomplished to obtain *in situ* measurements of undrained and remolded strength of clay and peat soils;

- Pressuremeter tests were used to obtain *in situ* stress-strain data, to enable calculation of soil shear modulus; and

- Down-hole compressional and shear wave velocity measurements were completed in a 100-foot-deep boring at each MSE wall location.

The last two of these special tests were accomplished specifically to obtain soil parameters for accurate modeling of MSE wall performance as discussed later in this report.

Soil samples were typically obtained in each boring at 2.5- to 5-foot-depth intervals. Each visible soil strata was individually sampled in the test pits and hand auger explorations.

Soil samples were visually classified in the field, in general accordance with the Standard Practice for Description and Identification of Soils (ASTM D 2488; see Figure 3). The classification is based on describing the density or consistency of the soil, moisture content, color, and gradation. Where present, organic material or debris was also noted.

Results of the explorations and field tests are presented in data reports, which are listed in the bibliography at the end of this report. (See for instance: AGI 1996 and 1998, CivilTech 1997 and 1998, HWA Geosciences 1998, and Hart Crowser 1999a, 1999b, 2000b, 2000d, 2000f, 2000j, 2000n, 2001b, and 2001j).

Laboratory Testing

Soil samples were delivered to Hart Crowser's laboratory in Seattle and logged into the sample tracking system. Hart Crowser's laboratory is currently certified by the Army Corps of Engineers to accomplish geotechnical testing on Corps' projects.

Upon receipt in the laboratory, the visual classification prepared in the field was checked under more controlled conditions, and samples were selected for testing. Moisture content was determined for most of the samples, and representative samples were selected for tests such as plasticity, gradation, strength, or compressibility.

Testing was accomplished in general accordance with the ASTM methods that are listed in Table 2.

All laboratory test results were reviewed by a Hart Crowser engineer, who prepared the data reports, summarized information for specific soil units, and compared results with properties estimated or reported by others for similar soils. In-house technical memoranda were prepared in some cases to summarize and document specific test results, (e.g., Hart Crowser 2001i and Appendix D in Hart Crowser 2000k).

3.2 Geologic Overview

For purposes of designing the Third Runway embankment and retaining walls, site geologic conditions can be divided into three areas of interest: a) relatively soft or loose surficial soils; b) dense or hard glacially overridden soils; and c) location and flow of shallow groundwater. Bedrock is quite deep and is not an explicit part of design except as it relates to potential earthquakes (discussed later).

Surficial Soils

Soils underlying the proposed Third Runway embankment typically consist of up to about 20 feet of loose to medium dense sandy soil with varying amounts of silt or clay, interbedded (or overlain) with soft to stiff sandy silt, clay, peat, and fill. Figure 3 summarizes the system we used to classify these soils and serves as a key to the exploration logs presented in other Third Runway project reports (Hart Crowser 1999a, 1999b, 2000b, 2000d, 2000f, 2000j, 2000n, 2001b, and 2001j). The surficial soils generally present at the Third Runway site included the following components, although not all these types are present at all locations.

Topsoil. Topsoil, consisting of a loose mixture of silt and sand with roots and other organic material, was intermittently encountered in our explorations, ranging from about 1/2 to 1 foot thick, where it was encountered.

Pre-Construction Fill. Existing fill, consisting of a loose to medium dense, variable mixture of silty or clayey sand and gravel, was encountered in some locations, typically associated with prior site use, including paved streets and residential housing. Fill is generally absent in the low-lying portions of the site adjacent to the creeks and wetlands. Most of the fill is less than 1 foot thick but occasionally varies up to 10 or more feet in thickness. The density and granular nature of the fill materials resembles the recessional outwash deposits described below, and the fill is sometimes difficult to distinguish from the outwash.

Alluvial Deposits Consisting of Interlayered Silt, Clay, Sand, and Peat. Alluvial deposits are sediments associated with Miller Creek or Walker Creek. These soils occur mainly in the low-lying areas to depths of up to about 15 feet.

The consistencies of the clay and silt deposits vary widely from soft to stiff or hard, and these soils generally contain sand fractions ranging up to about 30 percent by weight. Typically these clays and silts are low in plasticity, see Figure 4.

The alluvial sands are generally loose to medium dense, and range from non-silty to very silty or clayey (i.e., up to about 50 percent fines [particle sizes less than 0.074 mm]).

Peat was encountered in portions of some wetlands located near the west central part of the embankment, and in the north part of the embankment, both areas near to Miller Creek. Both surficial and shallow buried peat deposits were encountered. Buried deposits tend to be medium stiff to stiff, whereas the surficial peat exhibited consistencies in the very soft to soft range. Buried peat deposits were encountered at depths ranging from about 3 to 10 feet and varied in thickness between about 1 to 6 feet. Peat deposits near the ground surface varied in thickness between a few inches and about 2 feet.

Colluvium and Recessional Outwash. These soils generally consist of medium dense to dense, slightly silty to silty, slightly gravelly to gravelly sand.

Colluvium refers to soils that have been displaced by erosion or other natural processes on slopes subsequent to their original deposition. Recessional outwash overlies the glacial till, and overlies the advance outwash where the glacial till has been eroded. Thickness of the colluvium and recessional deposits varies over the site, but is generally less than 20 feet. These deposits vary in gradation over relatively short distances, and are intermittent or absent where alluvial materials are located.

Glacially Overridden Soils

Glacial Till. Glacial till soils observed at the site consist of dense to very dense, slightly gravelly to gravelly, silty to very silty sand. In general, glacial till differs from the overlying recessional soils by having a higher silt content and much higher density.

Glacial till is generally encountered within 10 to 20 feet of the ground surface, on the upper (eastern) part of west-facing slope on the west side of the existing airfield. The glacial till was not encountered in the explorations in downslope areas to the west, where the explorations terminated in advance soils. Springs and seeps occur along the western edge of the glacial till due to both perched water and interflow above the glacial till horizon as well as groundwater seepage from the aquifer in the underlying advance sands.

Advance Deposits. Underlying the glacial till are soils that were deposited in advance of glaciation and subsequently overridden. These advance soil deposits consist of dense to very dense, slightly silty, slightly gravelly to gravelly sand, with local interbeds of very stiff to hard silty or clayey soils. In general, but not always, the advance deposits can be distinguished from the glacial till by lower silt or clay content.

Groundwater

Shallow groundwater flows through the fill, colluvium, and alluvial soils, including seepage perched on the glacial till and on silty or clayey zones of the soils noted above. Seepage varies seasonally.

Shallow groundwater within the advance outwash soils and perched water in the overlying soil units combines to produce the "Shallow Regional Aquifer" in low lying areas adjacent to Miller Creek and Walker Creek. The Port has been monitoring water levels in this area for several years (1994 to date for some of the wells installed for the Third Runway), to assess the potential effect of embankment construction on base flow to these creeks and their tributary wetlands.

Shallow groundwater elevation contour maps have been developed and presented in several reports dealing with different parts of the project (Hart Crowser 1999c, 2000b, 2000f, and 2001j).

STIA also overlies two other aquifers that are considerably deeper and are used for water supply (AGI 1996).

An accompanying memorandum prepared for the Corps (Hart Crowser 2001l) discusses hydrogeology of the region and modeling to evaluate the effect of the Third Runway embankment on groundwater recharge and surface water hydrology.

3.3 Selection of Soil Parameters for Use in Analyses

The field and laboratory test results were reviewed to determine appropriate values for input to the geotechnical engineering analyses. Conservative test values were typically selected for use in the stability analyses, based on inspection of the range of data collected. Table 3 shows values of soil parameters used for different soil units in the stability analyses. Additional information on parameters used in the deformation analyses is presented in Hart Crowser (2000i).

Parameter values used in the geotechnical analyses were conservatively selected based on the range of results measured. Examples of this are illustrated on the figures described below.

- Figure 5 shows the range of drained friction angles measured over the range of embankment confining pressures (up to about 12 tons per square foot). Values were typically well above the 32 degree value used in analyses (see Table 3) especially at lower confining pressures.

- Figure 6 shows the undrained strength ratio (undrained shear strength normalized with respect to effective overburden pressure) used in our analyses, compared to undrained strength test results for the Third Runway project, and values reported by others for various soil types (Ladd 1986).

- Figure 7 shows the range in values for coefficient of consolidation, c_v, measured for silt and clay soils encountered in our borings. The design value used for analysis of pore pressures at the end of construction (EOC) is below most of the measured values, which results in conservative estimates of the rate of consolidation.

- Where possible, laboratory test measurements for parameters such as undrained strength, fines content, and consolidation coefficient were compared to field test measurements with the CPT, and field exploration data were used to define the areas where specific soils parameters were applicable.

Results of the laboratory tests are presented in data reports and memoranda, (See for instance: AGI 1996 and 1998, CivilTech 1997 and 1998, HWA

Geosciences 1998, and Hart Crowser 1999a, 1999b, 2000b, 2000d, 2000f, 2000j, 2000n, 2001b, and 2001j).

4.0 METHODS OF GEOTECHNICAL ANALYSIS

A number of geotechnical analyses have been completed for design of the Third Runway embankment and retaining walls, specifically including 1) stability of the embankment slopes and MSE walls; and 2) deformation, or movement, of the slopes and MSE walls, for both steady state and seismic conditions. These two types of analyses are discussed in this report because they pertain directly to the question of potential off-site impacts that is of interest to the Corps. (Other types of analyses such as settlement of the embankment, or infiltration and groundwater effects of the embankment, are discussed in Hart Crowser 2000g, Appendix C in Hart Crowser 2000o, and Hart Crowser 2001l).

4.1 Stability Analyses

Limit equilibrium stability analyses were used to evaluate design of the embankment fill, to design the extent of subgrade improvements, and to check the MSE wall reinforced zones. The AASHTO code specifies that both static and seismic analyses should be accomplished, and specifies target factors of safety that should be achieved. (Note, the Port used the same approach for "end of construction" analyses, which is not specified by AASHTO, but was appropriate to include for some soil conditions at the site.)

Table 4 lists the target factors of safety for limit equilibrium analyses used for the Third Runway. For comparison, Table 4 also shows the target factor of safety criteria used by the Corps of Engineers for comparable analyses of levees, as presented in EM 1110-2-1913 (Corps 2000).

Hart Crowser primarily used the program SLOPE/W (Geo-Slope 1998) for limit equilibrium analyses. We checked its performance by comparing analyses on specific MSE embankment sections to analyses using another well-documented program: UTEXAS3 (Hart Crowser 2001b).

To date 30 representative cross sections of the Third Runway embankment and retaining walls were analyzed using limit equilibrium analyses. Additional sections may be selected for further analysis depending on work in progress. Hart Crowser analyzed five to eight sections for each of the three MSE walls, and eight other sections to represent different areas of the 2H:1V embankment slopes. The sections used for analyses were selected to evaluate the range in subgrade conditions and embankment/wall geometries for the Third Runway project as a whole.

Figure 8 shows how soil strata are depicted for stability analysis of a typical embankment slope that is being checked for a potential failure surface; dozens of potential failure surfaces were analyzed for each cross section. In each case where the result did not meet or exceed the target factor of safety, the design was modified and the analysis was repeated until the target was met.

The analysis cases used for the Third Runway are described below:

- End of Construction (EOC) refers to the analysis of stability related to build-up of excess pore pressures in fine-grained soils in the embankment fill or subgrade, as construction proceeds. In cases where analyses using "worst case" unconsolidated, undrained (UU) strength parameters for foundation soils produced factor of safety values below the target level, stability was reanalyzed using more realistic partially consolidated strength properties. Our partially consolidated analysis used a spreadsheet model to calculate changes in subgrade strength due to pore pressure development and dissipation. Pore pressures were calculated as a function of the construction fill placement rate and measured thickness of silt and clay subgrade soils in different parts of the site. Target factor of safety for the EOC condition for MSE walls is 1.3.

- EOC analyses also included analysis of the range of excess pore pressures observed in previous construction with fine-grained embankment fill. Analysis of the Third Runway embankment for the pending Phase 5 construction with the maximum pore pressure values reported in the literature for embankments more than 200 feet high produced factors of safety of 1.3 or greater (Clough and Snyder 1966). We anticipate similar results would be achieved for future stages of embankment design. Hart Crowser is also using EOC analyses to check temporary cut slopes for the subgrade improvement excavations.

- Steady-state refers to the stability of the embankment under long-term conditions (i.e., with gravity loading but not seismic). Soil strength values used in these limit equilibrium analyses included the effect of strength gain due to consolidation from embankment construction, so a higher factor of safety is expected for some soils compared to the EOC condition. AASHTO allows the factor of safety for this condition to be either 1.3 or 1.5 depending on importance of the wall. Target factor of safety for MSE walls subject to steady state conditions for the Third Runway project is 1.5.

- Seismic stability analyses consisted of pseudo-static limit equilibrium type analyses, to conform to AASHTO criteria (AASHTO 1996-2000). AASHTO requires the target factor of safety for seismic conditions to be at least 1.1, which is the value used by the Port. The seismic hazard analysis used to obtain representative ground motions is described below in Section 4.2, (see also Hart Crowser 1999d, 2000e, and 2001a).

- For preliminary analyses, Hart Crowser used a value of 0.16 for the pseudo-static horizontal load vector in the limit equilibrium analyses. The initial value of 0.16 used for the pseudo-static load was half the peak horizontal acceleration (PHA) obtained from the averaged results of one-dimensional ground motion analysis (PROSHAKE) for embankment heights of 40 and 160 feet. Final design used half the PHA from the two-dimensional QUAD4 analyses discussed below, where this value was greater than 0.16.

  Hart Crowser used the consolidated undrained soil strength for cohesive soils (silts/clays) for the pseudo-static stability analysis (and the FLAC analysis discussed below) to account for the combined effect of both strength increase due to higher strain rate and potential strength reduction due to cyclic shaking.

  Minimum target factor of safety for the seismic (pseudo-static) stability specified by AASHTO is 1.1. For some areas, the analyses produced factors of safety between 1.0 and 1.1 for small potential failure surfaces near the toe of the fill or shallow raveling type zones on the upper surface of embankment slopes. In these instances, Hart Crowser verified the target factor of safety was met for deeper potential failure surfaces and relied on deformation analyses discussed below to verify there was no potential for progressive failure (i.e., potential for shallow raveling to lead to more extensive instability).

- Post-liquefaction stability analyses utilize reduced soil strength to represent the strength loss that occurs in some soils when excess pore pressures develop due to seismic shaking. Details of the liquefaction trigger analysis and estimation of post-liquefaction residual strength are discussed below in Section 4.3 (also see Hart Crowser 2001d). The target factor of safety for the post-liquefaction residual strength analyses was 1.1.

The limit equilibrium analyses were accomplished for both global stability and compound stability for the MSE walls. "Global stability" refers to analysis of potential instability due to failures below and behind the reinforced zone of the MSE walls, as shown on Figure 9. "Compound stability" refers to analysis of potential stability that extends through the reinforced zone as well as behind or below it (see Figure 10). In each analysis, a wide range of potential failure surfaces was examined, including circular surfaces, wedge-shaped surfaces, and irregular surfaces.

Limit equilibrium analyses were initially accomplished to estimate the spatial limits of subgrade improvement that might be needed using an assumed geometry for the reinforced zone behind the MSE walls (Hart Crowser 2000g). Additional analyses were accomplished for the 2H:1V embankment (Hart Crowser 2000o) and for the MSE walls using the reinforced zone geometry presented in RECo's 30 percent plans (Hart Crowser 2000m and 2001i). Limit equilibrium analyses for final design are currently in progress. For some of these analyses we are also considering the effect of using different backfill materials with higher strength values to potentially reduce the extent of subgrade improvements for particular sections, while still meeting performance standards.

MSE Wall Design Analyses

Section 5 of this report provides a summary of the MSE design process for the Third Runway; this subsection summarizes conventional limit equilibrium slope stability analyses that were utilized to check and/or modify the MSE design. Other forms of limit equilibrium analyses were also used by RECo for internal design of the reinforced zone for each of the Third Runway MSE walls in accordance with AASHTO code.

Design of MSE walls for the Third Runway is required to satisfy all of the following criteria:

1. Design requirements in the AASHTO code for MSE walls (AASHTO 1996-2000);

2. RECo in-house criteria, which include results of both theoretical and empirical methods of analysis, and performance criteria based on construction of similar walls; and

3. Verification that RECo's design meets the target factor of safety criteria for both global and compound stability (as described above); and

4. Verification that the proposed design will meet acceptable deformation criteria.

Table 5 summarizes geotechnical design requirements for the Third Runway MSE walls (for more detail see Hart Crowser 2000h). As noted above, the final design satisfies the strictest criteria from both RECo and AASHTO.

There is considerable similarity between the Third Runway design based on the AASHTO code requirements and the design criteria used by the Corps of Engineers for design of retaining walls, as presented in the engineering manual EM 1110-2-2502 (Corps 1989). Table 6 shows the Corps design criteria for retaining walls. The Corps criteria are very nearly the same as the Third Runway criteria presented in Table 5, with two minor exceptions:

- AASHTO allows the factor of safety for bearing capacity to be 2.0 on the basis of a detailed geotechnical analysis, while the Corps requires a value of 3.0. Analysis by Hart Crowser indicated the bearing capacity factor of safety for the Third Runway MSE walls exceeds the minimum value specified by the Corps.

- In addition, the sliding analysis specifically for walls on bedrock required by the Corps (see Note 3 in Table 6) is not applicable for the Third Runway, because the Third Runway walls are not founded on bedrock.

Except for the bedrock criterion that is not relevant, the design used for the Third Runway MSE walls meet or exceed comparable criteria used by the Corps (1989).

4.2 Deformation Analyses

Dynamic deformation analyses were used to assess performance of the Third Runway embankment and MSE walls by calculating how much movement would be produced by the design level shaking. The deformation analyses provide an independent check of the adequacy of the subgrade improvements, which were designed using the limit equilibrium analyses.

Two types of deformation model were used: a Newmark analysis and the finite difference model FLAC.

Newmark Analysis

Review by the ETRB identified reliance on pseudo-static analyses as one area where the Port could improve its design over the AASHTO requirements and recommended that a Newmark deformation analysis also be used.

The Newmark analysis method calculates displacements that will occur when the acceleration due to seismic shaking exceeds the level referred to as the yield acceleration (which is the acceleration that would produce a factor of safety of 1.0 in a pseudo-static analysis) (Newmark 1965). For this analysis, Hart Crowser used successive pseudo-static limit equilibrium analyses (accomplished with

Slope/W) to determine the yield accelerations for potential failure surfaces. In all cases we checked 10 or more potential failure surfaces for each of several cross sections. A two-dimensional site response program, QUAD4, was used to calculate seismic acceleration for each of these potential failure masses, using one or more acceleration time histories. Displacements were calculated by double integration of the motion during the times when acceleration produced by the time history exceeds the yield acceleration value.

Figure 11 illustrates a typical distribution of potential failure surfaces for the Newmark analysis of a MSE wall section, and the corresponding tabulated values of the yield acceleration k_y and maximum seismic acceleration k_{max}. We used both direct integration of the time history to estimate deformation, as well as the simplified approach using a k_y/k_{max} ratio as described by Makdisi and Seed (1978), since different magnitudes of deformation were produced by these methods for some of the sections. In most cases evaluated to date, the analysis showed negligible displacements (<0.1 foot). Subgrade improvements are being re-evaluated for two sections that had horizontal displacements of 1 to 2 feet.

Where the Newmark analysis displacements exceeded negligible values, Hart Crowser is accomplishing more detailed deformation analysis using the FLAC program. The Newmark analysis is also being used to check on some embankment sections to assess whether potential shallow surficial sloughing or small zones of potential instability (indicated by the pseudo-static limit equilibrium analysis) could lead to progressive raveling.

FLAC Analysis

The computer modeling program FLAC is being used to evaluate the seismic response and deformation of the Third Runway embankment and MSE walls. FLAC is an advanced tool for seismic analysis that is being used to confirm and supplement the conclusions from the more conventional analyses.

FLAC provides a good means to display results of stress-strain analysis using the finite difference method. The FLAC model helps illustrate the mechanisms of deformation, which generally verify the limit equilibrium analyses. (Lack of consistency between results of the two methods would be an indication of the need for further analysis of a particular section, if this were to occur.)

FLAC has been extensively used by others for dynamic analysis of earth structures, including some comparison of FLAC results with centrifuge models and in some cases with the effects of real earthquakes. Examples in engineering literature include: Inel, Roth, and C. de Rubertis 1993, Lee 1997, Makdisi, Wang, and Edwards 2000, Bathurst and Hatami 1998 and 1999, and Roth et al. 1993.

The Third Runway design team is using FLAC analysis techniques that have been demonstrated effective by research completed at the University of Washington that includes use of FLAC for both static and seismic analyses of MSE wall performance. The University of Washington research demonstrates the reasonableness of FLAC analyses for seismic analysis of MSE walls based on comparison with shaking table and centrifuge test results.

The finite difference mesh used in the FLAC model is "built" incrementally to provide a realistic estimate of stresses and deformations due to the weight of the fill. A "time history" of earthquake motion provides the basis for calculating additional stresses and deformations to assess the effect of design level earthquake shaking on the proposed embankment and MSE walls. The FLAC program provides both graphic and tabulated output, which can be used for further analysis, (for example see Hart Crowser 2000m and 2001g).

Figure 12 shows an example of the maximum horizontal displacement calculated for preliminary analysis of a representative section of the west MSE wall. The displacement contours indicate that the top of the wall would have a permanent displacement of about 10 inches resulting from the earthquake design motion (discussed below). The calculated vertical deformations are much less than the horizontal displacement. Another part of this same analysis provides designers with a tabulation of the maximum stress in the MSE reinforcing strips used in this section (see for example Hart Crowser 2001g).

FLAC model results are used to check predicted deformation vs. performance goals for the MSE walls. As needed, the reinforced zone or the subgrade improvements can be modified and the analysis repeated to see how performance (displacement or stress) is affected. An acceptable design for each section is obtained by comparing the results of both limit equilibrium and deformation models. Use of FLAC enables the Port to estimate wall movement and stresses in the reinforcing for a wide range of conditions from construction through performance in various size earthquake events, a capability that is not equally available from alternative computer models.

The FLAC analyses used for the Third Runway are above and beyond conventional design practice for MSE walls, i.e., the AASHTO code, which only requires pseudo-static analyses, used by the Port. However, the use of deformation-based analyses is gaining wide acceptance because of limitations in other types of analyses. Use of FLAC by the Port's design team provides an increased level of understanding regarding the MSE walls performance both during construction and in service.

4.3 Seismic Basis of Design

Input for both QUAD4 and FLAC is in the form of a record of motion, which is developed from an earthquake acceleration record selected to represent a "design level earthquake." This section discusses the basis for selecting the design level earthquake.

The Third Runway embankment and MSE walls are being designed to perform well during and after earthquake shaking that has a 10 percent probability of exceedence in 50 years, or an average return period interval of once in 475 years. Seismic events of this frequency are commonly used for design of many structures such as commercial buildings and highway bridges. This is the same basis of design return period that the Port of Seattle has used for other significant structures at STIA, such as the South Terminal Expansion Project currently under construction.

The process used to determine the magnitude of the seismic basis of design event began with a Probabilistic Seismic Hazard Assessment (PSHA). The PSHA utilizes thousands of analyses (for different source-site distances, magnitudes, and earthquake characteristics [such as the effects of fault type], and attenuation relationships) to produce a probability based uniform hazard spectra that represents potential earthquake effects on the site (Hart Crowser 1999d, 2000e, and 2001a).

Several ground motions have been utilized for the Third Runway analysis to cover the range of earthquake shaking characteristic of the design level event. These motions, designated A, B, C and D, include one motion that is deterministically based, to specifically assess motion on the most significant local fault, the Seattle Fault.

Initial design analyses used the model PROSHAKE to complete a one-dimensional site response analysis. The average peak horizontal acceleration (PHA) from this analysis was used to provide input to a) the pseudo-static analyses used to evaluate global and compound stability; and b) the MSE design analyses accomplished by RECo. The AASHTO design method includes PHA in a Mononabe Okabe-type analysis for determination of lateral earth pressures.

Subsequent Third Runway design analyses used the program QUAD4 to complete two-dimensional site response analysis for representative embankment and MSE wall sections. The QUAD4 analysis was used to obtain the following:

- Seismic cyclic shear stresses at different locations, to assess potential for liquefaction below or adjacent to the embankment;

- Maximum acceleration (K_{max}) to be used in the Newmark analysis; and

- Verification that the preliminary PROSHAKE-derived PHA values used in the pseudo static analyses are conservative, or to provide PHA (K_{max}) values for re-analysis.

Finally, QUAD4 was used to compare the effects of the different ground motions and to produce the input ground motion for the FLAC analyses.

Although not a formal part of selecting the seismic basis of design for the Third Runway, the design team made a careful assessment of conditions at the project site (and performance of local MSE walls) following the February 28, 2001, Nisqually earthquake (see Hart Crowser 2001c, 2001e, and 2001f). No adverse effects of that earthquake were observed in the native soils on the Third Runway fill placed prior to that time.

4.4 Liquefaction Analysis

"Liquefaction" refers to the temporary reduction in shear strength that occurs in some soils as a result of development of excess pore pressures that develop in an earthquake. Identification of the conditions that will trigger liquefaction and calculation of the post-liquefaction soil strength are important parts of the geotechnical analysis affecting stability and deformation of the Third Runway embankment and MSE walls.

Potential liquefaction is a consideration for some areas of the native soils that underlie the proposed embankment, including portions of the MSE walls. The effected soils are saturated, predominantly granular, and typically loose to medium dense. Some areas of silty or clayey soils were also found to be susceptible to liquefaction, based on screening using the "Chinese Criteria" as modified by the Corps (Kramer 1996).

Trigger Liquefaction

Determining the susceptibility of soils to loss of strength due to liquefaction is referred to as the "trigger liquefaction" analysis. Trigger liquefaction analysis is based on a recent update to the state of the art method (Youd et al. 2001). The trigger liquefaction analysis compares *in situ* soil characteristics at the Third Runway site with soil parameters that have been found to indicate liquefaction, (Seed and Harder 1990 and Idriss 1998).

The Third Runway embankment incorporates an underdrain over much of its base area, including the areas below the three MSE walls. The main purpose of the underdrain is to prevent development of any excess pore pressures within the embankment such as might develop from saturation due to infiltration or filling over existing surface seeps. Drainage provided by the underdrain and the dense compaction of the embankment fill protect the embankment itself from liquefaction. The potential occurrence of liquefaction is limited to some areas of existing native soils. The purpose of the liquefaction analysis is to identify the areas where subgrade improvement is needed to mitigate potential instability, or excessive deformation, due to liquefaction.

Details of the liquefaction analysis for the Third Runway are presented in Hart Crowser (2000k and 2001d). More recent analyses have incorporated cyclic shear stresses calculated with QUAD4.

The trigger liquefaction analysis uses a factor of safety of 1.25 to account for small increases in pore pressures that may have some effect on strength. This safety factor is separate from, and in addition to, achieving the target factor of safety in the previously discussed limit equilibrium analyses. The trigger liquefaction analysis provides the values of SPT required to trigger liquefaction which are then compared with SPT values measured at the site (Hart Crowser 2000k and 2001h). The adjustment in N-values is based on well-documented procedures (Youd et al. 2001). We also evaluated CPT data for prediction of liquefaction at the Third Runway site.

Soil conditions were evaluated for more than 25 cross sections that were selected to represent the range in subgrade and embankment/MSE wall configuration. For each cross section, the adjusted N-values required to trigger liquefaction were compared to the SPT and CPT data. Potentially liquefiable zones were delineated, and the residual strength was estimated using SPT data. The post-liquefaction stability was analyzed with limit equilibrium methods to determine the extent of subgrade improvement needed to meet the target factor of safety, as previously discussed.

Residual Strength Calculation

Large ground failures and deformations resulting from liquefaction have only been documented to occur when adjusted SPT N-values are 15 or less (Seed and Harder 1990 and Idriss 1998). However, our analysis suggested that liquefaction could potentially occur for some soil conditions at the site corresponding to N-values up to around 30. To address the potential effect of this on stability, the Third Runway design team used a soil behavior-based extrapolation of the documented residual strength of soils that have liquefied. We calculated the residual strength using corrected SPT blow counts $(N_1)_{60\text{-}CS}$ by extrapolating the residual strength curve (Idriss 1998). While there is no theoretical basis for limiting residual strength increases based on extrapolation of these curves, we limited and capped the extrapolated residual strength to 1,200 psf, corresponding to $(N_1)_{60\text{-}CS} = 24$.

For each MSE wall or embankment cross section, the N-values which fell below the threshold value of $(N_1)_{60\text{-}CS}$ were tabulated and residual strength calculated for each soil unit. Each cross section evaluation included consideration of changes in soil parameters observed in explorations on each side of the cross section, along with the maximum groundwater level at each well (see Hart Crowser 2001j). The range of interpolation for each cross section varied, depending on how closely spaced the sections are to one another. We looked for consistent soil units that extended from one cross section to the next, as well as for local variations that distinguished one section from another.

Residual strength values were selected for liquefiable soil units. The residual strength values used for analysis were selected to provide a reasonable lower bound, looking at the range and variation of specific SPT values in each unit, where a soil unit was identified on the basis of continuous soils of similar gradation, density, and saturation. We used the lower third value of the range for residual strength in each unit if the data showed much scatter; where there was no significant scatter, we used the mean value of residual strength for the analysis.

Finally, estimated residual undrained strength values were checked to make sure they do not exceed the drained shear strength for the same type soil. The stability analyses used the lower value of either the estimated residual strength or the drained shear strength.

5.0 MSE WALLS

This section discusses why MSE walls were selected for the Third Runway, and specific design steps used for the Third Runway MSE walls.

5.1 Background

During preliminary stages of design, the Port of Seattle reviewed eight different types of retaining walls and more than 60 wall/slope combinations to identify the best means of limiting the embankment impact to Miller Creek, Walker Creek, and adjoining wetlands. The Port of Seattle selected MSE walls as the best alternative for the project based on seismic performance, constructability, historical performance, and cost-effectiveness (HNTB et al. 1999). The selection of MSE technology was confirmed via a peer review by Shannon & Wilson (1999).

After selection of MSE walls as the best alternative to limit embankment impacts to creeks and wetlands, the Port of Seattle consulted with in-house staff and experts at the University of Washington and the Washington State Department of Transportation to determine appropriate criteria for selection of an MSE wall design engineer for the Third Runway MSE walls. A formal request for qualifications was published through the mailing lists from two MSE trade associations, the Geosynthetics Materials Association and the Association for Metallically Stabilized Earth.

The Port's design team received and reviewed nine submittals from prospective designers of the Third Runway MSE walls. The Port selected RECo USA, the North American subsidiary of Terre Armee International (TAI), based on their recent experience with MSE walls of similar height and layout as those planned for the Third Runway. RECo/TAI has been responsible for design and construction of more than a dozen walls more than 90 feet in height, including two that are about the same height as the maximum wall height proposed for the Third Runway. Upon selection of RECo as the MSE wall designer, they were assimilated into the design team with HNTB and Hart Crowser. Construction of the MSE walls will be accomplished by a general contractor with components specified by the design team, and manufactured from any supplier.

5.2 Design of MSE Walls

The following steps were utilized in the progressive design and analysis of MSE walls for the Third Runway.

- An initial layout of MSE walls was developed to fit within the embankment geometry and minimize or avoid impacts to wetlands as much as possible.

- The design team met to review and discuss the design parameters, loads and details (geotechnical recommendations for design are presented in Hart Crowser 2000h). Over a period of several weeks, the design team worked through regular teleconferences to review proposed design criteria and reached consensus on the basis for design, including structural, mechanical, and aesthetic details.

- Using initially assumed reinforcement geometry, limit equilibrium analyses were used to verify that design could satisfy the AASHTO code (AASHTO 1996-2000) and other design requirements for conditions at the Third Runway site.

- Analysis of preliminary sections was used to assess the need for subgrade improvement in order to satisfy stability and allowable settlement criteria.

- Initial wall design, including length, depth, and density of MSE reinforcing was developed by RECo, based on the design criteria and RECo design computations. RECo evaluated internal stability needs for a 100-year performance period addressing reinforcement durability, pullout and tensile capacity. External stability was evaluated for sliding and overturning.

- RECo submitted design plans showing type, size, and location of MSE wall and reinforcing components, for review by HNTB and Hart Crowser. RECo developed hand calculations to check and document the results of computer-based analyses. These calculations along with RECo's project Quality Assurance Plan were reviewed by Hart Crowser and HNTB. Written comments were submitted to document recommendations (HNTB 2001).

- Hart Crowser checked global and compound stability of the initial RECo design sections, and accomplished initial deformation analyses (Hart Crowser 2000m, 2001g, and 2001i).

- Hart Crowser is now checking deformation of the MSE sections with the Newmark analysis. Sections with a) the lowest factor of safety; or b) largest deformation from the Newmark analysis have been selected for further deformation analysis with FLAC.

- Architectural and structural issues continue to be addressed in light of geotechnical needs. These include arrangement of wall facing details to accommodate vertical settlement joints; wall panel thickness; reinforcing strip lengths; number of reinforcing strips per panel; tier elevation; top treatments, etc.

At various stages of the analyses outlined above, modifications were made as necessary to change the extent of subgrade improvement and/or the length or depth of the MSE reinforcing zone. After review by the design team, recommended subgrade and/or MSE modifications were incorporated into the design in an iterative manner (Hart Crowser 2001g and 2001i).

6.0 CONSTRUCTION CONTROL

As previously mentioned, the Port plans to use "subgrade improvement" to mitigate areas of soft or loose soils that affect stability or deformation. This includes areas of compressible soils, soils with low shear strength, and soils that are subject to liquefaction. The anticipated subgrade improvements range from about 15 to 20 feet below the existing ground surface, based on information from the existing borings.

The Port reviewed nine different methods for subgrade improvement (Hart Crowser 2000g) and selected two preferred alternatives: 1) removal and replacement with compacted structural fill, or 2) stone columns. Relative feasibility, including the degree of ground improvement, constructability, quality assurance, and cost were considered for the Third Runway project., as well as potential post-construction effects on base flow to Miller Creek and adjacent wetlands (Hart Crowser 2000p).

Final selection of the removal and replacement method was made by the Port after stone column field tests were accomplished as part of the Phase 4 construction in 2001. These tests included collection of SPT and CPT data, accomplished before and after installation of more than 100 stone columns in four test patterns. The tests indicated that it would be difficult to obtain the same degree of construction quality assurance with the stone column method as with the remove and replace method. The remove and replace method was selected because it would achieve better construction reliability.

The Port has successfully monitored embankment construction to date, using the same type of soils and methods of construction that are planned for the remainder of the embankment. Construction specifications allow different types of soil materials to be used in different parts of the embankment, with appropriate moisture content limits, lift thickness, and compacted density specified to achieve a consistent quality earth fill. Compaction control and other fill quality tests are based on Federal Aviation Administration specifications (P-152) that have been modified to reflect local soil conditions.

Backfill for the subgrade improvement areas will utilize very densely compacted granular fill, compacted to 95 percent of the modified Proctor maximum density per ASTM method D 1557. The Port utilizes full-time construction inspection and services of a testing lab, field results are reviewed by both HNTB and Hart Crowser to verify conformance to the specifications.

7.0 REFERENCES

7.1 Bibliography of Third Runway Reports Related to Design

AGI 1996. Baseline Groundwater Study, Appendix Q-A, Final Environmental Impact Statement, Proposed Master Plan Update, Sea-Tac International Airport, SeaTac, Washington. January 3, 1996.

AGI 1998. Geotechnical Design Recommendations Phase 1 Embankment construction, Third Runway Project, Sea-Tac International Airport, Seatac, WA. January 22, 1998.

CivilTech 1997. Geotechnical Report, South 154th Street/156th Way Relocation, SeaTac International Airport, SeaTac, Washington. October 27, 1997.

CivilTech 1998. Supplemental Geotechnical Report, South 154th Street/156th Way Relocation, SeaTac International Airport, SeaTac, Washington. September, 1998.

FAA 1996. Final Environmental Impact Statement for Proposed Master Plan Update Development Actions at Seattle-Tacoma International Airport. US Department of Transportation, Federal Aviation Administration.

FAA 1997. Final Supplemental Environmental Impact Statement for Proposed Master Plan Update Development Actions at Seattle-Tacoma International Airport. US Department of Transportation, Federal Aviation Administration.

HNTB, Hart Crowser, and Parametrix 1999. Draft Evaluation of Retaining Wall/Slope Alternatives to Reduce Impacts to Miller Creek Embankment Station 174+00 to 186+00, Third Dependent Runway, Sea-Tac International Airport. April 12, 1999.

HNTB 2001. Memorandum: Seattle-Tacoma international Airport, Review Comments on 30% Retaining Wall Design Package. March 30, 2001.

HWA Geosciences 1998. Geotechnical and Dewatering Evaluation, miller Creek Relocation, SeaTac Airport, Washington. June 8, 1998.

Hart Crowser 1998. Resource Evaluation and Conceptual Development for Borrow Areas 3 and 4, Third Runway Project, Seattle-Tacoma International Airport. December 17, 1998.

Hart Crowser 1999a. Use of Taxiway Soils in 1999 Fill, Third Runway Embankment, Memorandum to Jim Thomson, HNTB, March 4, 1999.

Hart Crowser 1999b. Subsurface Conditions Data Report, 404 Permit Support, Third Runway Embankment, Sea-Tac International Airport, SeaTac, Washington, July 1999.

Hart Crowser 1999c. Geotechnical Engineering Report, 404 Permit Support, Third Runway Embankment, Sea-Tac International Airport, SeaTac, Washington, July 9, 1999.

Hart Crowser 1999d. Sea-Tac Airport Third Runway, Probabilistic Seismic Hazard Analysis Results, Memorandum to Jim Thomson, HNTB, October 8, 1999.

Hart Crowser 2000a. DRAFT Geotechnical Engineering Report, Phase 3 (Y2000) Fill, Third Runway Embankment, Sea-Tac International Airport, January 27, 2000.

Hart Crowser 2000b. DRAFT Subsurface Conditions Data Report, North Safety Area, Third Runway Embankment, Sea-Tac International Airport, SeaTac, Washington, March 20, 2000.

Hart Crowser 2000c. Memorandum: Bench Study for Slope Embankment; March 31, 2000

Hart Crowser 2000d. DRAFT Subsurface Conditions Data Report, South MSE Wall and Adjacent Embankment, Third Runway Project, Sea-Tac International Airport, SeaTac, Washington, April 7, 2000.

Hart Crowser 2000e. DRAFT Memorandum: Seismic Basis of Design, Third Runway. April 10, 2000.

Hart Crowser 2000f. DRAFT Subsurface Conditions Data Report, West MSE Wall, Third Runway Embankment, Sea-Tac International Airport, SeaTac, Washington, June 2000.

Hart Crowser 2000g. Preliminary Stability and Settlement Analyses, Subgrade Improvements, MSE Wall Support, Third Runway Project, June 2000.

Hart Crowser 2000h. DRAFT Memorandum: Geotechnical Input to MSE Wall and Reinforced Slope Design, Third Runway Embankment. August 21, 2000.

AR 014227

Hart Crowser 2000i. DRAFT Memorandum: Use of Advanced Testing Data, Sea-Tac Third Runway Project. August 28, 2000.

Hart Crowser 2000j. DRAFT Subsurface Conditions Data Report, Additional Field Explorations and Advanced Testing, Third Runway Embankment, Sea-Tac International Airport. September 5, 2000.

Hart Crowser 2000k. Memorandum: Methods and Results of Liquefaction Analyses, Third Runway Embankment, Sea-Tac International Airport. September 7, 2000.

Hart Crowser 2000l. Memorandum: Third Runway - South MSE Wall Analysis Station 147 + 25. November 3, 2000.

Hart Crowser 2000m. DRAFT Stability Review of RECo 30 Percent Design, Third Runway Embankment Project. November 9, 2000.

Hart Crowser 2000n. DRAFT Subsurface Conditions Data Report, Phase 4 Fill, Third Runway Embankment, Sea-Tac International Airport. November 29, 2000.

Hart Crowser 2000o. DRAFT Geotechnical Engineering Report, Phase 4 Fill, Third Runway Embankment, Sea-Tac International Airport. December 4, 2000.

Hart Crowser 2000p. Memorandum: Proposed MSE Wall Subgrade Improvements, Seattle-Tacoma International Airport. December 8, 2000.

Hart Crowser 2001a. Memorandum: Additional Information on Seismic Design, Job Number 4978-30, dated January 25, 2001.

Hart Crowser 2001b. DRAFT Memo to File, Response to Sea-Tac 3rd Runway Project Technical Review Board Meeting on November 16, 17, and 18, 2000, dated February 28, 2001.

Hart Crowser 2001c. Post-Earthquake Reconnaissance and Photo Documentation, Job Number 4978-06, dated March 2, 2001.

Hart Crowser 2001d. Draft Memorandum: Revised Methods and Results of Liquefaction Analyses, Job Number 4978-30, dated March 5, 2001.

Hart Crowser 2001e. Memorandum: MSE Wall Performance During Earthquake, Job Number 4978-19, dated March 26, 2001.

Hart Crowser 2001f. Memorandum: Post-Earthquake Reconnaissance and Photo Documentation, Job Number 4978-06, dated March 26, 2001.

Hart Crowser 2001g. Draft Memorandum: Reinforcement Stresses Calculated in FLAC, Third Runway Project, Sea-Tac, Washington, Job Number 4978-30, dated April 18, 2001.

Hart Crowser 2001h. Draft Memorandum: Proposed Liquefaction Procedure under Walls and Slopes, Job Number 4978-30, dated May 14, 2001.

Hart Crowser 2001i. Memorandum: Revised Stability Review of RECo 30 Percent Design - South Wall, Third Runway Embankment Project, Job Number 4978-30, dated June 7, 2001.

Hart Crowser 2001j. Subsurface Conditions Data Report, Phase 5 Fill and Subgrade Improvement, Third Runway Embankment, Sea-Tac International Airport SeaTac Washington. September 21, 2001.

Hart Crowser 2001k. DRAFT Geotechnical Engineering Analyses and Recommendations, Third Runway Embankment, Phase 5, Third Runway Embankment, Sea-Tac International Airport SeaTac Washington. September 26, 2001.

Hart Crowser 2001l. Memorandum: Wetland Hydrology and the Third Runway Embankment Fill. October 30, 2001.

Shannon & Wilson 1999. Peer Review, Seattle-Tacoma International Airport Third Runway, Seattle, Washington. Letter to HNTB, March 27, 1999.

7.2 Additional Technical References Related to Design

AASHTO 1996-2000. "Standard Specifications for Highway Bridges," 16th Edition, 1996, with current interim addenda through 2000. American Association of State Highway and Transportation Officials.

ASCE 1983. Advisory Notes on Lifeline Earthquake Engineering Report Prepared by the Technical Committees of the ASCE Technical Council on Lifeline Earthquake Engineering. American Society of Civil Engineers.

ASTM 2001. Annual Book of ASTM Standards. Volume 04.08 and 04.09, American Society for Testing and Materials.

Bathurst, R.J., and K. Hatami 1998. "Seismic Response Analysis of a Geosynthetic-Reinforced Soil Retaining Wall", Geosynthetics International, V. 5 Nos. 1-2.

Bathurst, R. J., and K. Hatami 1999. Earthquake Response Analysis of Reinforced-soil Walls Using FLAC. Proceedings of the International FLAC Symposium on Numerical Modeling in Geomechanics.

CERF 1998. Guidelines for Evaluating Earth Retaining Systems. CERF Report No. 40334, Civil Engineering Research Foundation. March 1998.

Clough, G. W. and J. W. Snyder 1966. Embankment Pore Pressures During Construction. Technical Report No. 3-722, US Army Corps of Engineers Waterways Experiment Station, Vicksburg, Mississippi.

Corps 1989. U.S. Army Corps of Engineers, Engineering and Design, Retaining and Flood Walls. EM 1110-2-2502. September 29, 1989.

Corps 1995. Engineering and Design, Earthquake Design and Evaluation for civil works Projects. ER 1110-2-1806. July 31, 1995.

Corps 2000. Engineering and Design, Design and Construction of Levees. EM 1110-2-1913. April 30, 2000.

FHWA 1997. Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines, SA-96-071. Federal Highway Administration.

FHWA 2001. Mechanically Stabilized Earth Walls and Reinforced Slopes, Design and Construction Guidelines, FHWA NHI-00-043. March 2001.

Geo-Slope 1998. User's Guide SLOPE/W for stability analysis, Version 4. Geo-Slope international Ltd. Calgary, Alberta.

Idriss, I.M. 1998. Evaluation of liquefaction potential, consequences and mitigation and update. Presentation Notes, Vancouver Geotechnical Society Meeting February 17, 1998.

Inel, S., W.H. Roth, and C. de Rubertis 1993. "Nonlinear Dynamic Effective Stress Analysis of Two Case Histories," Proceedings of the Third International Conference on Case Histories in Geotechnical Engineering pp 1735-1741.

Kramer, S.L. 1996. Geotechnical Earthquake Engineering. Prentice Hall Inc.

Ladd, C.C. 1986. Stability Evaluation During Staged Construction. ASCE Journal of Geotechnical Engineering, V. 117, N.4. April 1991.

Lee, W.F. 1997. "Numerical Analysis of Instrumentation of a Geosynthetic Reinforced Wall," Industrial Fabrics Association International: Geosynthetics, Vol. 1.

Makdisi, F.I. and H.B. Seed 1978. Simplified Procedure for Estimating Dam and Embankment Earthquake-induced Deformations. ASCE Journal of Geotechnical Engineering Division, Vol. 104, No. 7, pp. 849-867.

Makdisi, F.I., Z-L Wang, and W.D. Edwards 2000. "Seismic Stability of New Exchequer Dam and Gated Spillway Structure," Proceedings of the Twentieth Annual USCOLD Lecture Series: Dam O&M Issues - The Challenge of the 21st Century.

Newmark, N.M. 1965. Effects of Earthquake on Dams and Embankments. Geotechniques. Geotechnique, Vol. 5, No. 2, June, 1965.

Pockowski, M. and J.M. Duncan 2000. Comparison of Computer Programs for Analysis of Reinforced Slopes. Virginia Tech Center for Geotechnical Practice and Research, December, 2000.

Roth, W.H. et al. 1993. "Upper San Fernando Dam 1971 Revisited". Annual Conference Proceedings of the Association of State Dam Safety Officials. D.W. Darnton and S.C. Plathby eds. Lexington, KY.

Seed, H.B. 1979. Considerations in Earthquake-Resistant Design of Earth and Rockfill Dams. Geotechniques, Vol. 29, No. 3, pp. 215-263.

Seed, R.B. and L.F. Harder 1990. SPT-based analysis of cyclic pore pressure generation and undrained residual strength, in J.M. Duncan ed., *Proceedings, H. Bolton Seed Memorial Symposium*, University of California, Berkeley, Vol. 2, pp. 351-376.

Youd, T.L., I.M. Idriss, R.D. Andrus, I. Arango, G. Castro, J.T. Christian, R. Dobry, W.D. Liam Finn, L.F. Harder Jr., M.E. Hynes, K. Ishihara, J.P. Koester, S.S.C. Liao, W.F. Marcuson III, G.R. Martin, J.K. Mitchell, Y. Moriwaki, M.S. Power, P.K. Robertson, R.B. Seed, and K.H. Stokoe II 2001. Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils. Journal of Geotechnical and Geoenvironmental Engineering, Vol. 127, No. 10, pp. 817-903. October 2001.

F:\Docs\Jobs\497806\ACOESummaryReport_2.4.doc

Table 1 - Summary of Explorations

Preliminary Evaluation & Environmental Assessment Phase

91 Borings (12 Monitoring Wells)

34 Test Pits

7 Vane Shear Tests

Final Design Phase

127 Borings (65 Monitoring Wells)

122 Test Pits

48 Cone Penetrometer Soundings

10 Vane Shear Tests

Notes:

1. Table includes explorations related to main embankment as well as for partial relocation of Miller Creek for the North Safety Area embankment construction, but does not include geotechnical studies for relocation of South 154th Street, borrow sites, or other parts of the Port of Seattle Capital Improvement Program. Hand auger explorations for wetlands delineation and shallow soil sampling not shown.
2. See Plates 1, 2, and 3 for location of explorations.

Table 2 - Laboratory Test Methods

Soil Classification	ASTM D 2488 (for visual identification only) and ASTM D 2487 (precise classification based on measured indices)
Classification of Peat	ASTM D 4427
Soil Moisture Content	ASTM D 2216
Grain Size Analysis	ASTM D 422
Atterberg Limits (Liquid Limit, Plastic Limit and Plasticity Index)	ASTM D 4318
One-dimensional Consolidation Test	ASTM D 2435
Consolidated Undrained Triaxial Test	ASTM D 4767
Unconsolidated Undrained Triaxial Test	ASTM D 2850
Direct Shear Tests	ASTM D 3080

Table 3 - Soil Parameters Used in Stability Analyses

Soil Type	Unit Weight in pcf	Drained Strength		Undrained Strength Parameters
		c' in psf	ϕ' in Degrees	S_u/σ_v' (a)
Existing Subgrade Soils				
Loose to medium dense Sand	125	0	32	-
Medium dense to dense Sand	130	0	35	-
Dense to very dense Sand	135	0	37	-
Glacial Till	130	250	40	-
Soft Peat or Organic Silt (c)	110	0	7 to 15	0.23
Medium stiff Silt/Clay (b)	115	0	30	0.23
Stiff to hard Silt/Clay (b)	115	0	30	0.23
Post Construction Soils				
Embankment Fill	135	0	35	-
Drainage Blanket	140	0	37	-
Improved Subgrade	135	0	35	-

(a) Undrained strength ratios were used for fine-grained soils based on CU triaxial results and are a function of confining pressure (σ_v'). For pseudo-static analyses, this value is assumed to reflect the combined effect of strength increase due to high rate of seismic loading and potential strength reduction due to cyclic loading.

(b) Undrained strength parameters were used for the end-of-construction cases, otherwise, drained strength properties were used.

(c) Drained friction angle for the peat was 15 degrees except at low confining pressure where a value of 7 degrees was used, see Hart Crowser (2001k).

AR 014234

Table 4 - Target Factors of Safety for Limit Equilibrium Analyses

Type of Analysis [1]	Target Factor of Safety Used for Third Runway MSE Wall Design	Target Factor of Safety Used by Army Corps of Engineers for Levees (EM 1110-2-1913, Corps 2000)
End of Construction	1.3	1.3
Steady State	1.5	1.4
Seismic	1.1	See note 2
Post-liquefaction	1.1	See note 2

Notes:

1. The Rapid Drawdown case used by the Corps is not applicable to the Third Runway because the Third Runway embankment does not retain water.
2. The Corps of Engineers does not specify a target factor of safety for seismic analysis. Reference to ER 1110-2-1806 (Corps 1995) indicates the Corps relies on procedures that include assessment of project hazard potential, potential earthquake motion and project features to determine design requirements for specific projects. This is essentially the same as the procedure used for the Third Runway as described in Section 4.3 and applied in the analyses described in Sections 4.1, 4.2, and 4.4.

AR 014235

Table 5 - Summary of Design Requirements for Third Runway MSE Walls   Sheet 1 of 2

5-1 - Static Stability Analysis [a]

	AASHTO 1996 - 2000 (Target F.S. or Other)	RECo Design Manual 1999 (Target F.S. or Other)
External Stability		
Sliding	≥1.5	≥1.5
Overturning	≥2.0	≥2.0
Eccentricity at Base	Not specifically stated	Not specifically stated
Bearing Capacity (for sliding and overturning)	≥2.0 (if justified by geotech analysis); ≥2.5 otherwise	≥2.0 (if detailed geotech info.); ≥2.5 (if general geotech info.)
Deep-Seated Stability (i.e., Global and Compound Stability)	≥1.3 (if soil param. based on lab tests); ≥1.5 otherwise	Not specifically stated
Internal Stability		
Pullout Resistance	≥1.5, where maximum friction angle of 34 deg. is used to calculate the horizontal force (if without the benefit of triaxial or direct shear testing to provide soil shear strength data)	Defaults to AASHTO, Interim 1998
Pullout Resistance[b]	$T_{max} \leq 0.55\ F_y$	$T_{max} \leq 0.55\ F_y$

5-2 - Seismic Stability Analysis [a]

	AASHTO 1996 - 2000 (Target F.S. or Other)	RECo Design Manual 1999 (Target F.S. or Other)
External Stability		
Sliding	≥1.1; include 100% of inertial force and 50% of dynamic thrust[c]	≥1.1
Overturning	≥1.5; include 100% of inertial force and 50% of dynamic thrust	≥1.5
Eccentricity at Base	Not specifically stated	Not specifically stated
Bearing Capacity (for sliding and overturning)	75% static (i.e., ≥1.5; include 100% inertial force and 50% of dynamic thrust [c]	Not specifically stated
Deep-Seated Stability (i.e., Global and Compound Stability)	≥1.1	Not specifically stated
Internal Stability		
Pullout Resistance	75% static; reduce F* to 80% static value; include internal inertial force[d]	Not specifically stated
Pullout Resistance	$T_{max} \leq 0.55\ F_y$	$T_{max} \leq 0.55\ F_y$

AR 014236

Table 5 - Summary of Design Requirements for Third Runway MSE Walls (cont'd) Sheet 2 of 2

5-3 - Comparison of Other Aspects of MSE Wall and Reinforced Slope Design Standards [a]

	AASHTO 1996 - 2000	RECo Design Manual 1999
MSE Embedment[e]	H/7 for 2H:1V slope in front of wall, where H is from top of wall at wall face to top of leveling pad	Same as AASHTO 1996
Horizontal Bench in Front of Walls Founded on Slopes	4 feet minimum width	3 feet minimum width
Calculation of Sliding for External Stability	Neglect passive resistance; include width and weight of wall facing in calculation of sliding/overturning	Not specifically stated
Leveling Pad Width	Designed to meet local bearing capacity needs and differential settlement between wall facing and backfill	Not specifically stated
Maximum particle size for reinforced backfill (see text for detailed discussion)	4 inches	6 inches
Friction Factor for Internal Reinforcement Design (backfill on ribbed steel strips)	$F^*_{max}<2.0$; $F^*_{max}<1.2 + \log C_u$, where C_u equals backfill uniformity coefficient. $C_u = 4$ for ribbed steel strips if tests are not available	Based on extensive pullout tests, but no values are specifically stated

5-4 - Comparison of Recommended Backfill Electrochemical Properties [a]

	AASHTO 1996 - 2000	RECo Design Manual 1999
Soil pH	5 to 10	5 to 10
Soil resistivity (at 100% saturation)	>3000 ohm-cm[f]	>3000 ohm-cm
Water soluble chloride content	<100 ppm	<100 ppm
Water soluble sulfate content	<200 ppm	<200 ppm
Organic content	1% max. (for material finer than No. 10 sieve)	Free of organics and other deleterious materials

a Note Third Runway MSE design is controlled by the "more strict" requirement when AASHTO and RECo are not the same. See also FHWA 1997 for criteria not specified by either AASHTO or RECo, such as base eccentricity (Hart Crowser 2000h).

b T equals "tension" and F_y equals "yield strength."

c Dynamic thrust determined by the pseudo-static Mononobe-Okabe analysis.

d F* is the friction factor variable, which is part of the reinforcement pullout analysis.

e MSE embedment is not a specific requirement of AASHTO or FHWA, but is provided as guidance for MSE constructed on fill.

f If soil resistivity is greater than or equal to 5,000 ohm-cm, the chlorides and sulfates requirement may be waived.

AR 014237

Table 6 - (from Corps 1989)

Retaining Wall Stability Criteria

Case No.	Loading Condition	Sliding Factor of Safety, FS	Shear Strength Test Required: Soil Foundation	Shear Strength Test Required: Rock Foundation[3]	Overturning Criteria Minimum Base Area in Compression: Soil Foundation	Overturning Criteria Minimum Base Area in Compression: Rock Foundation	Minimum Bearing Capacity Safety Factor
R1	Usual	1.5	(Q &/or S)[2,1]	Direct shear	100%[4]	75%[4]	3.0
R2	Unusual	1.33	(Q &/or S)[2,1]	Direct shear	75%[4]	50%[4]	2.0
R3	Earthquake	1.1	(Q)	Direct shear	Resultant within base	Resultant within base	>1.0

Notes

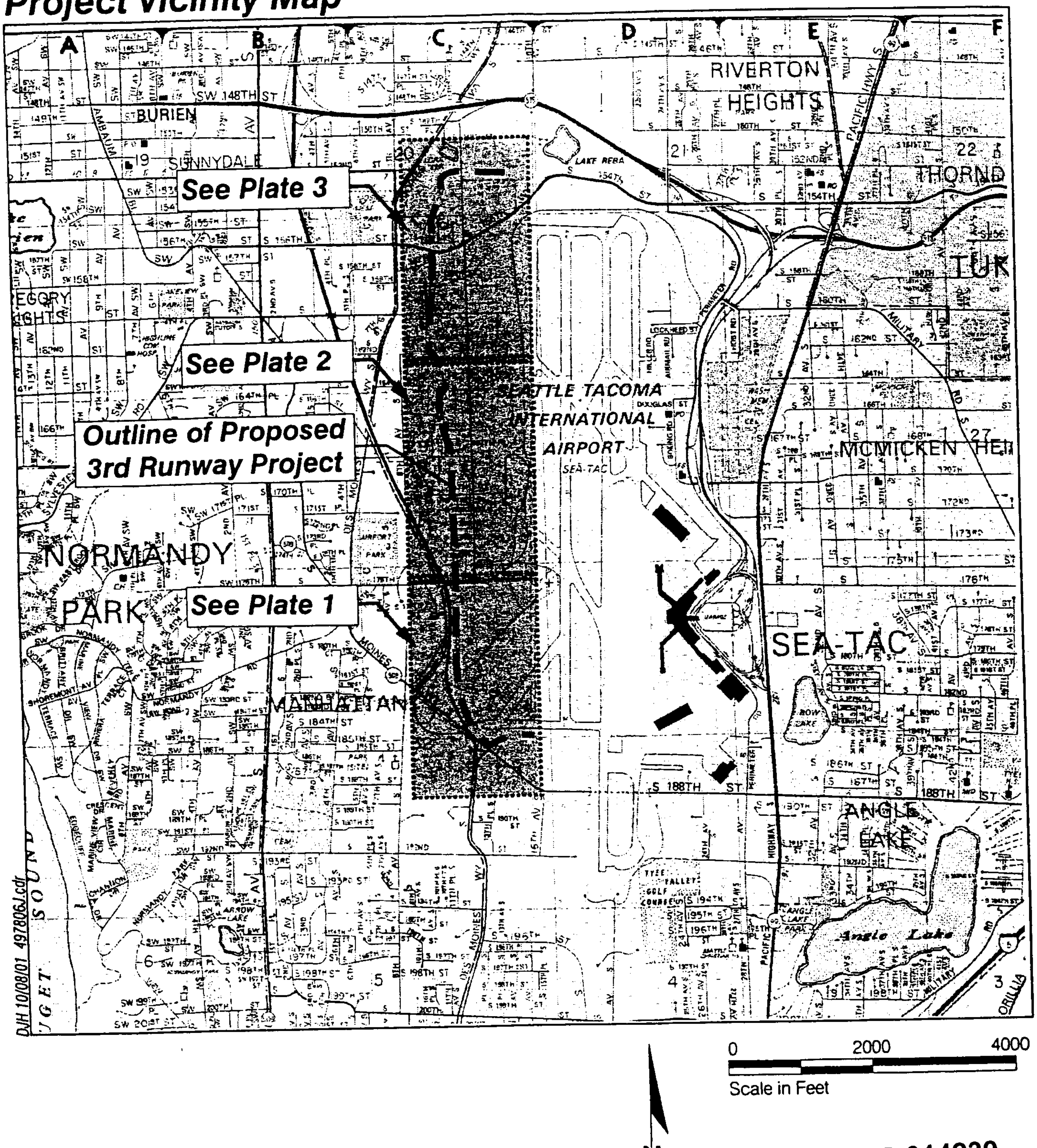
1. For soil foundations which are not free draining (permeability $< 10 \times 10^{-4}$ cm/sec), analyze for both Q and S strengths and design for the worst condition. For free-draining soil foundations (permeability $> 10 \times 10^{-4}$ cm/sec), analyze for S strengths only.

2. For construction loadings in Cases R1 and R2, use Q strengths when excess pore water pressure in the soil foundation is anticipated and S strengths when it is not anticipated.

3. The sliding analysis of a wall on rock should be based on the frictional resistance ($\tan \phi$) of concrete on rock or rock on rock. The values should be obtained from direct shear tests of pre-cut samples of concrete on rock and rock on rock, or direct shear tests of natural rock joints or bedding planes.

4. Less base area in compression than the minimum shown may be acceptable provided adequate safety against unacceptable differential settlement and bearing failure is obtained.

Project Vicinity Map



DJH 10/08/01 497806J.cdr

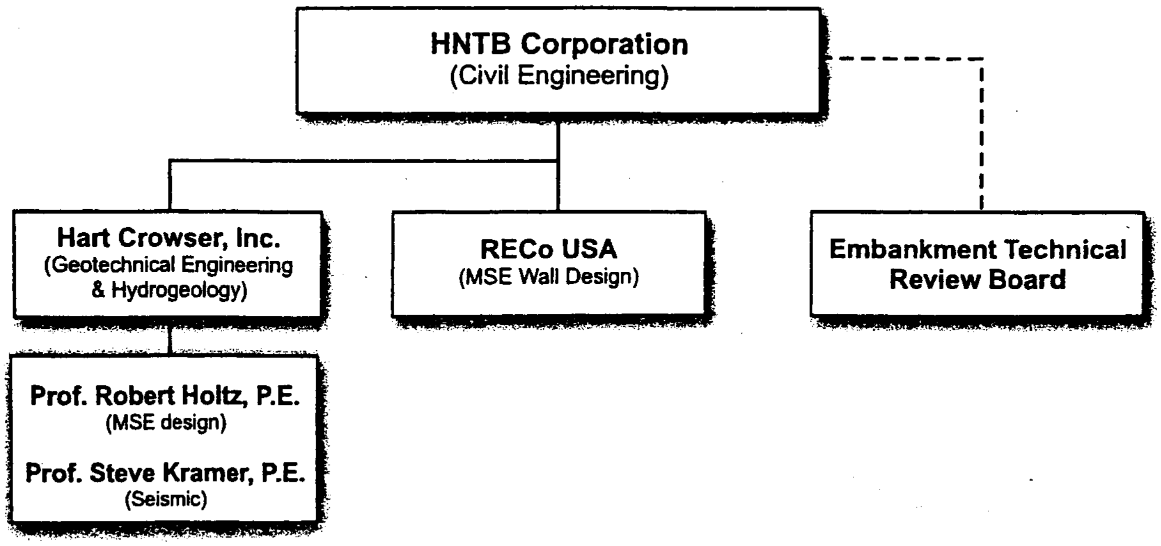
AR 014239

HARTCROWSER

J-4978-06 10/01

Figure 1

Organization Chart for Third Runway Embankment Design Team and Independent Review Board



AR 014240

hel 10/31/01 497806K.cdr

HARTCROWSER

J-4978-06 *10/01*

Figure 2

Soil Classification System and Key to Exploration Logs

Sample Description

Classification of soils in this report is based on visual field and laboratory observations which include density/consistency, moisture condition, grain size, and plasticity estimates and should not be construed to imply field nor laboratory testing unless presented herein. Visual-manual classification methods of ASTM D 2488 were used as an identification guide.

Soil descriptions consist of the following:
Density/consistency, moisture, color, minor constituents, MAJOR CONSTITUENT, additional remarks.

Density/Consistency

Soil density/consistency in borings is related primarily to the Standard Penetration Resistance.
Soil density/consistency in test pits is estimated based on visual observation and is presented parenthetically on the test pit logs.

SAND or GRAVEL Density	Standard Penetration Resistance (N) in Blows/Foot	SILT or CLAY Consistency	Standard Penetration Resistance (N) in Blows/Foot	Approximate Shear Strength in TSF
Very loose	0 - 4	Very soft	0 - 2	<0.125
Loose	4 - 10	Soft	2 - 4	0.125 - 0.25
Medium dense	10 - 30	Medium stiff	4 - 8	0.25 - 0.5
Dense	30 - 50	Stiff	8 - 15	0.5 - 1.0
Very dense	>50	Very stiff	15 - 30	1.0 - 2.0
		Hard	>30	>2.0

Moisture

Dry	Little perceptible moisture
Damp	Some perceptible moisture, probably below optimum
Moist	Probably near optimum moisture content
Wet	Much perceptible moisture, probably above optimum

Minor Constituents

	Estimated Percentage
Not identified in description	0 - 5
Slightly (clayey, silty, etc.)	5 - 12
Clayey, silty, sandy, gravelly	12 - 30
Very (clayey, silty, etc.)	30 - 50

Legends

Sampling Test Symbols

BORING SAMPLES

- Split Spoon
- Shelby Tube
- Cuttings
- Core Run
- * No Sample Recovery
- P Tube Pushed, Not Driven

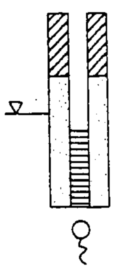
TEST PIT SAMPLES

- Grab (Jar)
- Bag
- Shelby Tube

Test Symbols

GS	Grain Size Classification
CN	Consolidation
UU	Unconsolidated Undrained Triaxial
CU	Consolidated Undrained Triaxial
CD	Consolidated Drained Triaxial
QU	Unconfined Compression
DS	Direct Shear
K	Permeability
PP	Pocket Penetrometer Approximate Compressive Strength in TSF
TV	Torvane Approximate Shear Strength in TSF
CBR	California Bearing Ratio
MD	Moisture Density Relationship
AL	Atterberg Limits
	Water Content in Percent
	Liquid Limit
	Natural
	Plastic Limit (NP=Non Plastic)
PID	Photoionization Detector Reading
CA	Chemical Analysis
DT	In Situ Density Test

Groundwater Observations



Surface Seal

Groundwater Level on Date (ATD) At Time of Drilling

Observation Well Tip or Slotted Section

Groundwater Seepage (Test Pits)

DJH 1=1 497806 A-1 Boring key.DWG

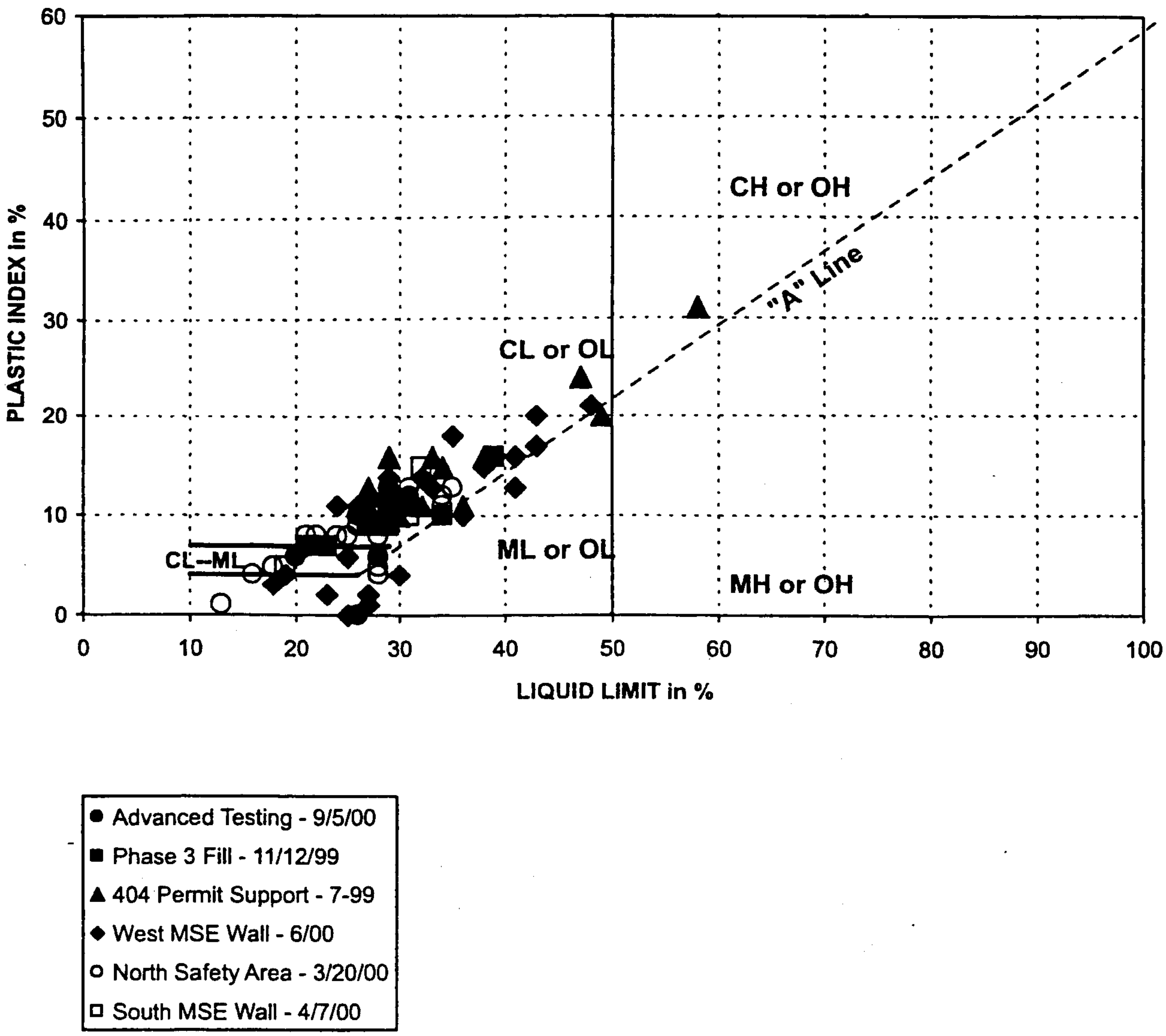
AR 014241

HARTCROWSER

J-4978-28 10/01

Figure 3

Soil Plasticity Summary Plot



AR 014242

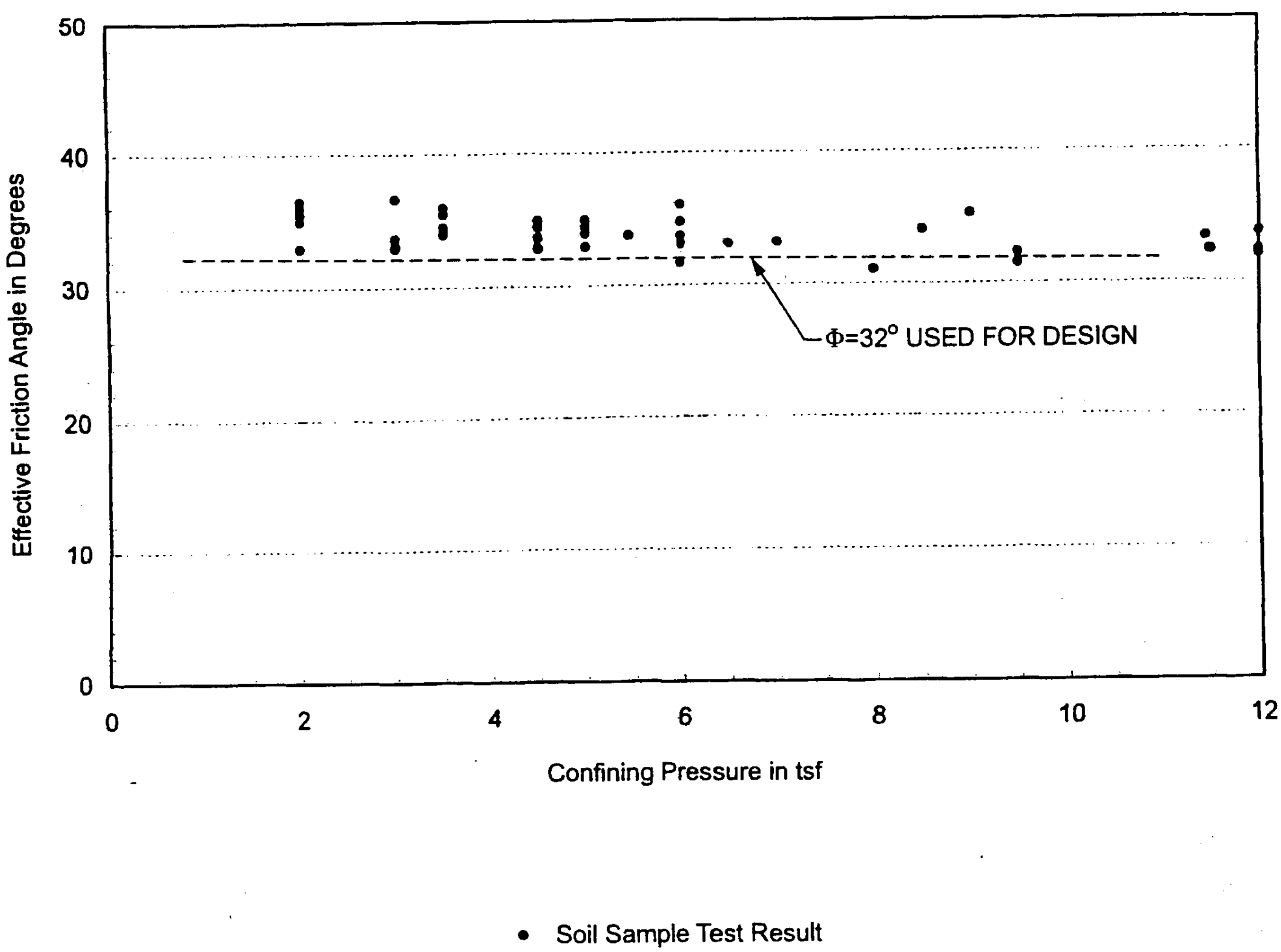
HARTCROWSER

J-4978-06 10/01

Figure 4

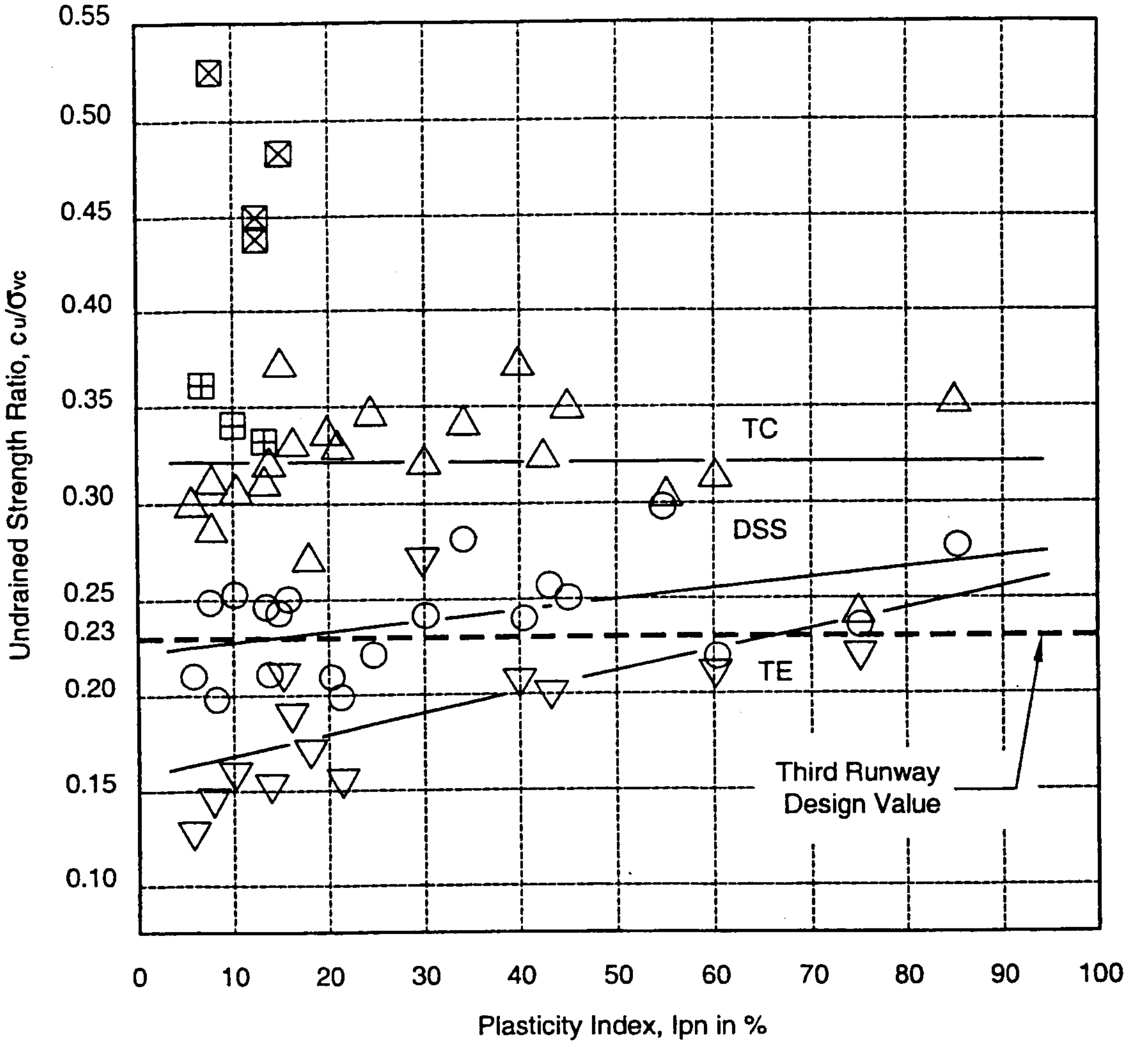
hel 10/31/01 497806L.cdr

Effective Friction Angle vs. Confining Pressure for Clays and Silts



AR 014243

Undrained Strength Ratio for Normally Consolidated Clays and Silts Compared to Design Value and Published Data



Legend:

⊞ Hart Crowser High-Pressure CU Triaxial Tests-2001

⊠ Hart Crowser CU Triaxial Tests-1999-2001

△ Triaxial Compression (TC) :qf

○ Direct Simple Shear (DSS) : τh

▽ Triaxial Extension (TE) :qf

Data from Ladd 1986

AR 014244

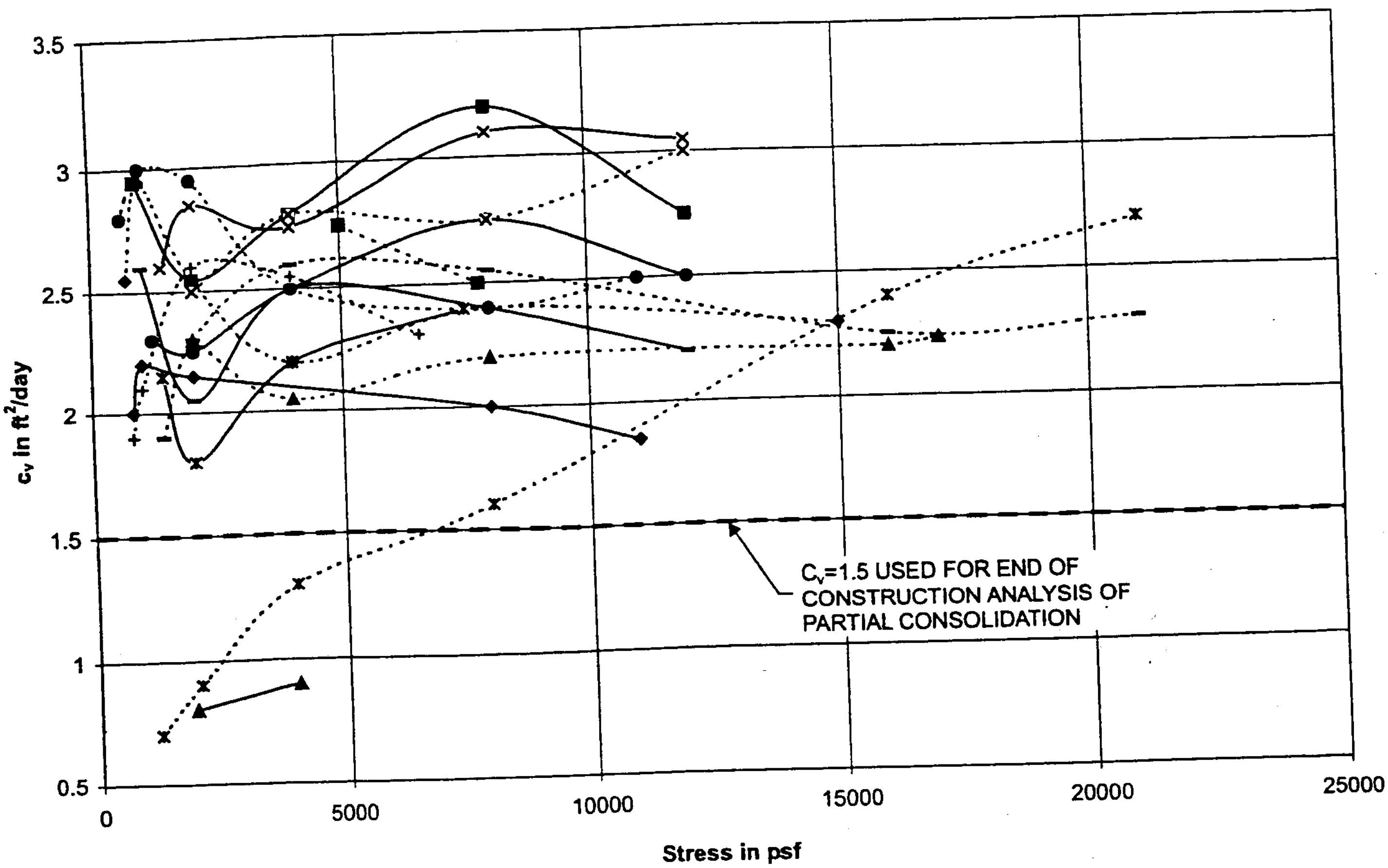
497806098.DWG CAS NOT TO SCALE

HARTCROWSER

J-4978-06 ***10/01***

Figure 6

Coefficient of Consolidation vs. Embankment Load Range



Sample Key

- B-44
- B-54A
- B-163
- B-164
- B-165A
- B-167
- B-169
- B-110
- B-111-S3
- B-111-S-6
- B-115
- B-118-S-2
- B-118-S-6
- B-132A
- B-142

Note:
The lower and upper stresses for each sample represent *in situ* and *in situ* + embankment load respectively, such that the results are applicable for the stress range during construction.

AR 014245

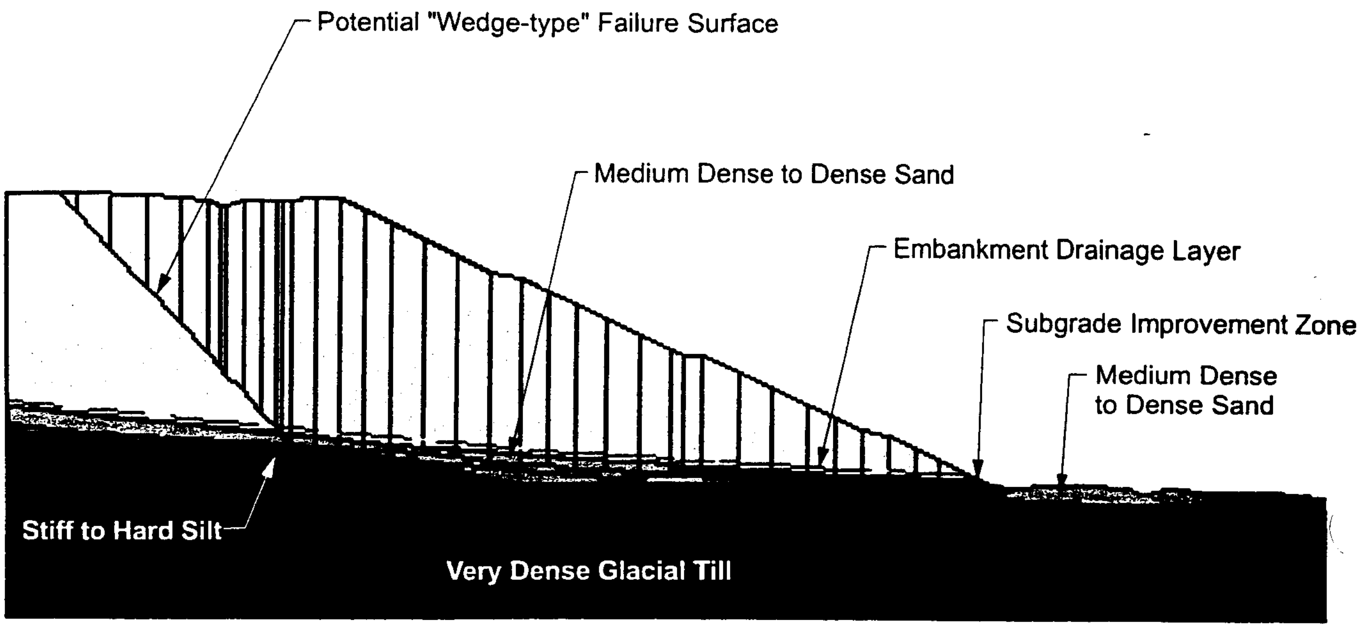
HARTCROWSER

J-4978-06 10/01

Figure 7

hel 10/31/01 4978O6M.cdr

Equilibrium Stability Analysis for a 2H:1V Embankment Section



Note:
This figure illustrates a typical limit equilibrium analysis using Spencer's method with the program SLOPE/W. Each stability analysis includes calculating factor of safety for dozens of such surfaces. The limits (width and depth) of the subgrade improvement zone are adjusted so the analysis proceeds until all potential failure surfaces meet the target factor of safety. Subgrade improvements are constructed to mitigate weak or compressible soil or to assure stability.

AR 014246

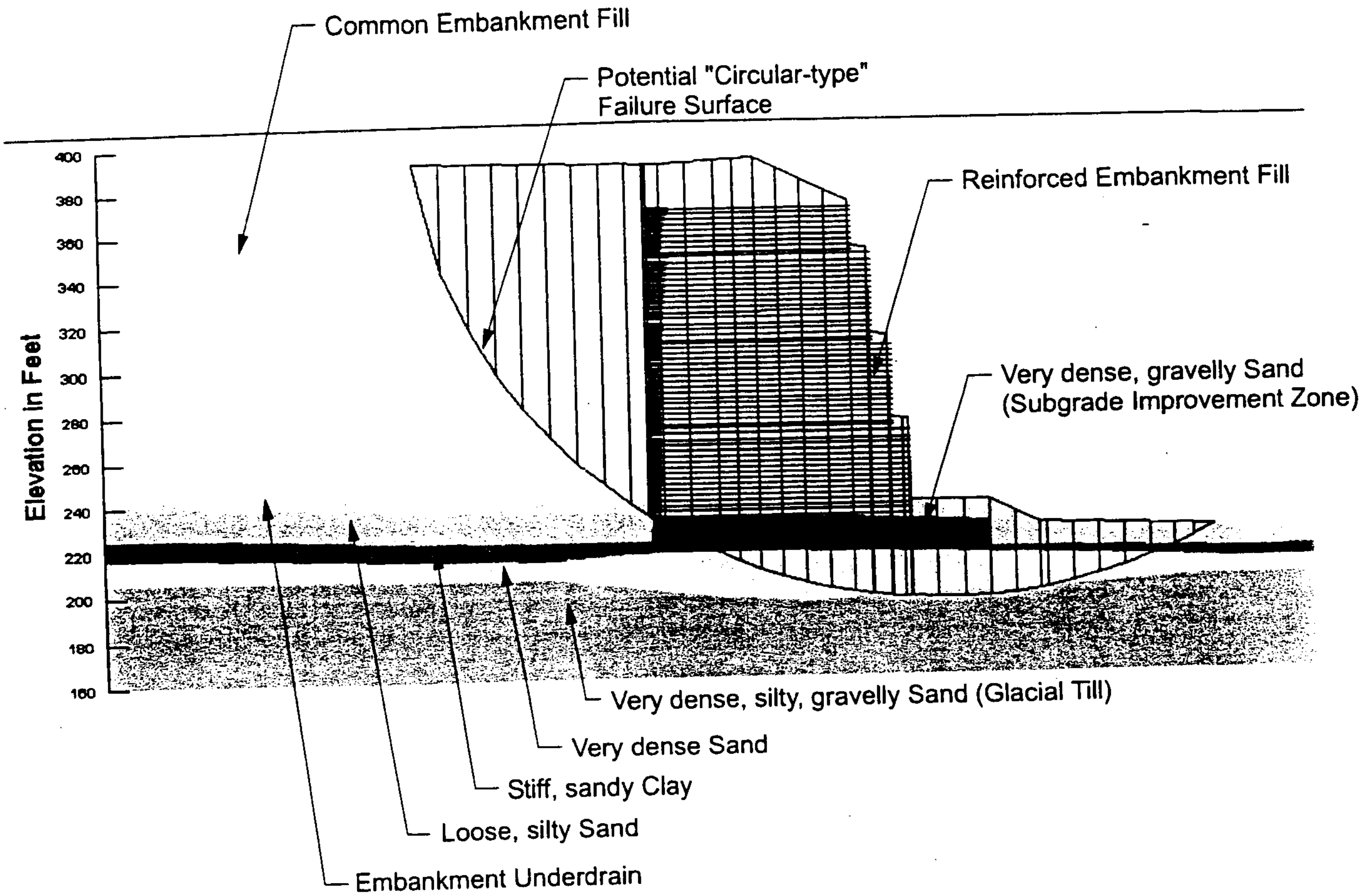
hel 10/31/01 497806N.cdr

HARTCROWSER

J-4978-06 ***10/01***

Figure 8

Global Stability Analysis for a West MSE Wall Section



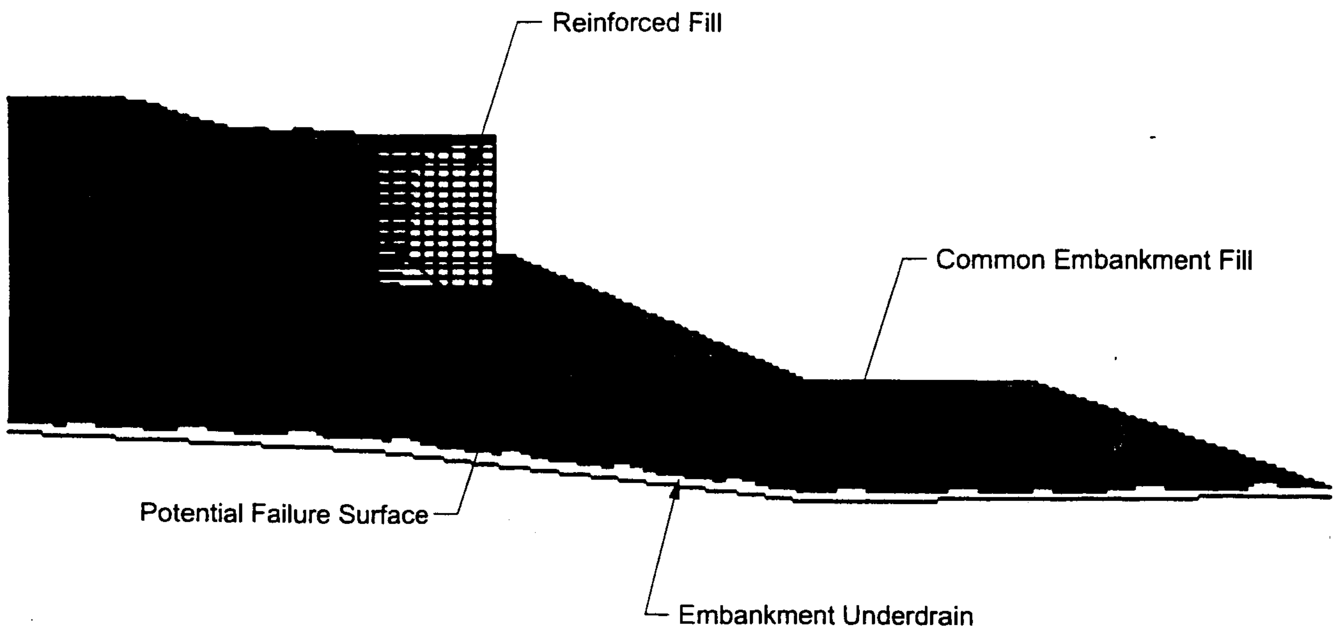
Note:
Global stability analysis is a type of limit equilibrium analysis that looks for potential failure surfaces that extend below and outside the MSE reinforcing.

AR 014247

HARTCROWSER
J-4978-06 *10/01*
Figure 9

hel 10/31/01 497806P.cdr

Compound Stability Analysis for a South MSE Wall Section



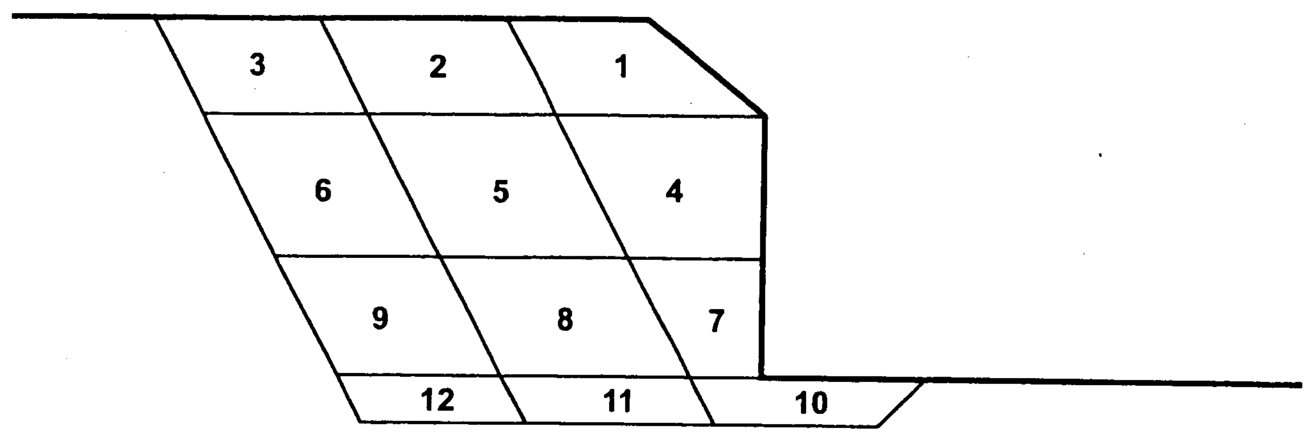
Note:
Compound stability analysis is a type of limit equilibrium analysis that looks for potential failure surfaces that extend through the soil reinforcing. As needed, the length, thickness, and/or depth of embedment of the MSE reinforcing can be adjusted for iterative analyses until all potential failure surfaces meet target factors of safety.

AR 014248

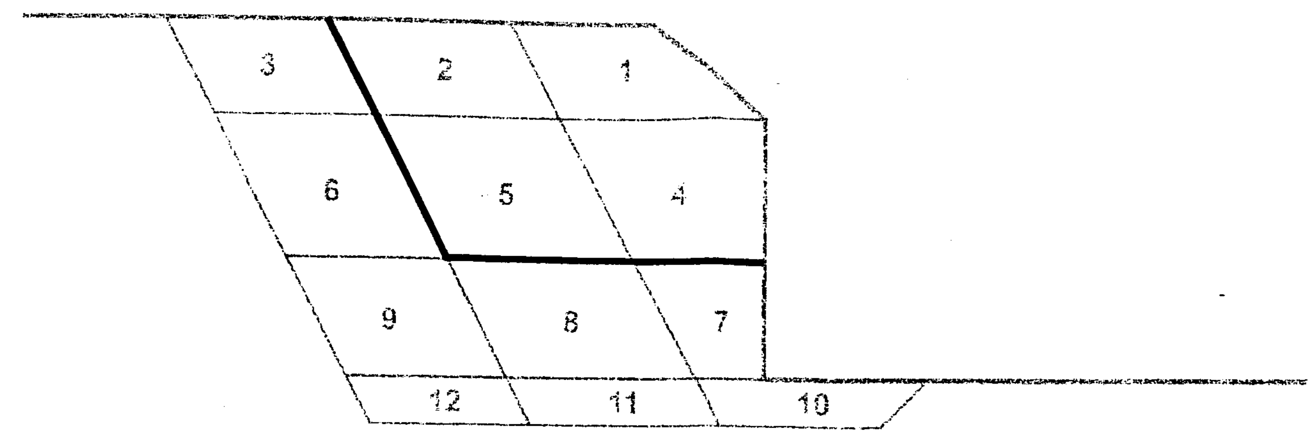
hel 10/31/01 4978060.cdr

HARTCROWSER
J-4978-06 *10/01*
Figure 10

Illustration of Newmark Deformation Analysis



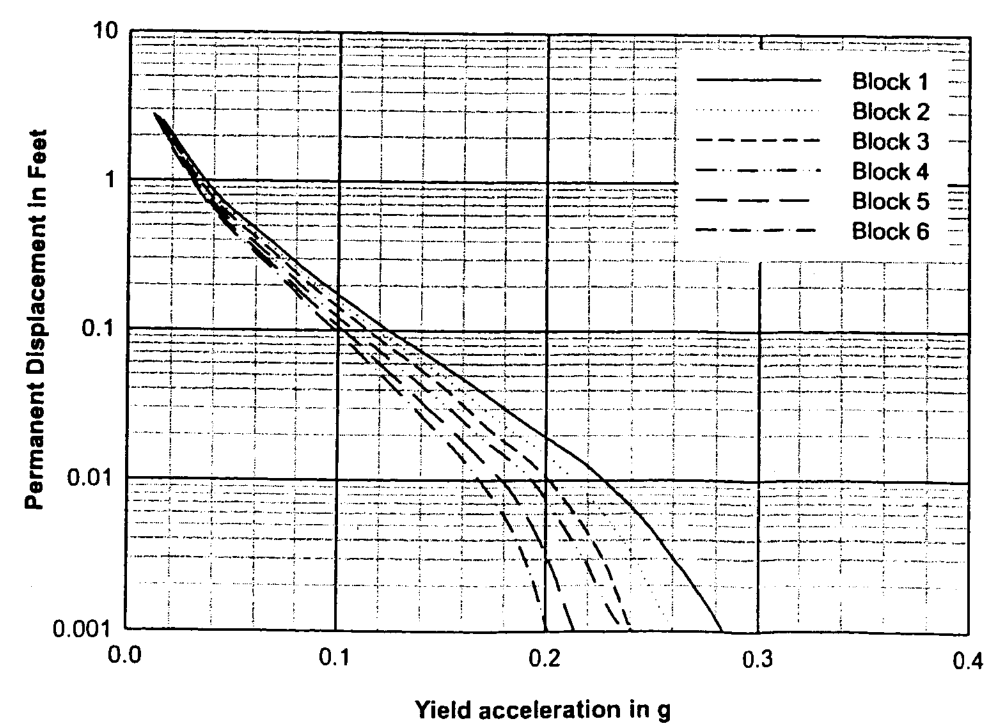
Note:
Illustration of numbering system used for identifying wedges. (For clarity, soil and wall details not shown)



Note:
Illustration of potential failure surface for Wedge 5 includes volume of potential failure wedges 1, 2, and 4.

Block ID	k_y	k_{max}
1	0.43	0.45
2	0.59	0.44
3	0.64	0.42
4	0.56	0.36
5	0.53	0.34
6	0.61	0.33
7	0.61	0.33
8	0.48	0.28
9	0.53	0.27
10	0.52	0.28
11	0.10	0.24
12	0.44	0.24

Note:
Seismic shaking produces displacement of a wedge when k_{max} exceeds k_y.



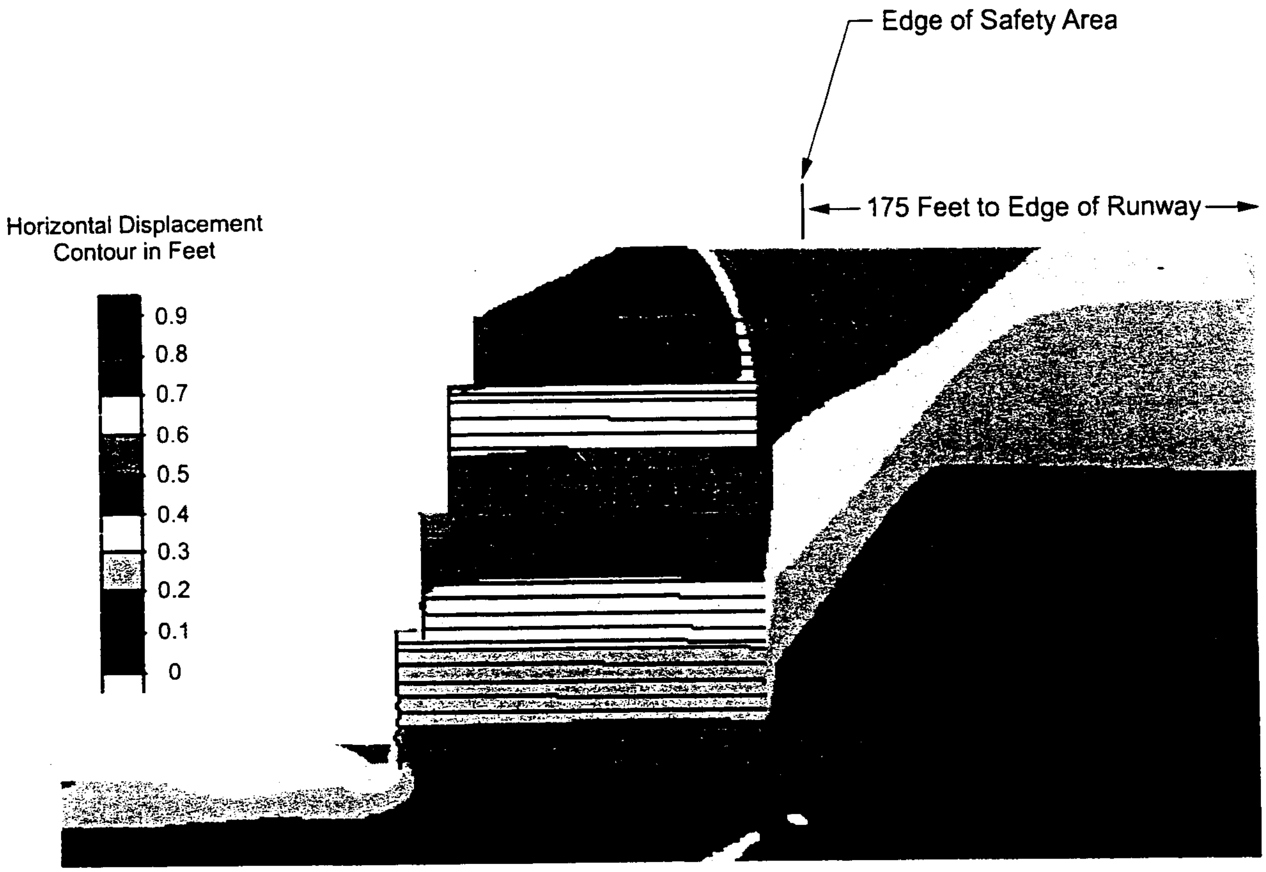
Note:
Permanent displacement varies depending on yield acceleration and k_{max} for each potential failure wedge.

AR 014249

HARTCROWSER
J-4978-06 *10/01*
Figure 11

hel 10/31/01 497806f.cdr

FLAC Model Deformation Analysis for a West MSE Wall Section



Note:
Illustration of horizontal ground displacement from FLAC model after design level earthquake shaking. Colors indicate approximate zones of uniform displacement. Details of soil horizons and subgrade improvement omitted from this figure for clarity.

hel 10/31/01 497806S.cdr

HARTCROWSER
J-4978-06 ***10/01***
Figure 12

C

GEOSYNTEC CONSULTANTS

1500 Newell Avenue, Suite 800
Walnut Creek, California 94596 • USA
Tel. (925) 943-3034 • Fax (925) 943-2366

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.

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.

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.

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

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.

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

/For

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

I:\WR0380\SeaTac Comment Letter.doc

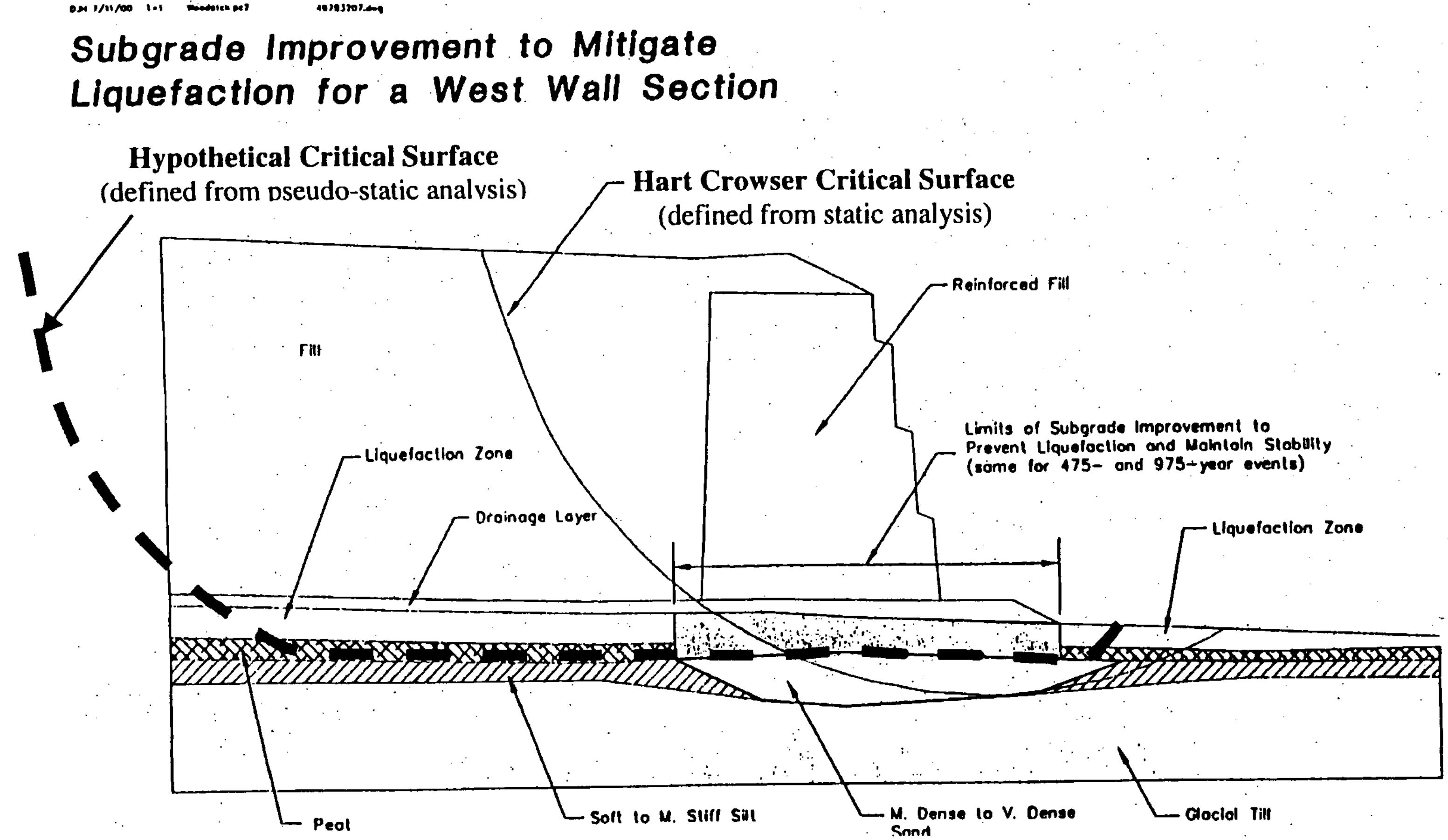


Figure 1: Hypothetical Seismic Stability Failure Surface

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.

"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.

"Seismic Basis of Design – Third Runway Project," Hart Crowser Memorandum, April 10, 2000.

"Geotech Review" – Two page handwritten commentary on ISW Lagoon #3 project geotechnical engineering report by Zipper Zeman Associates, Inc. by Washington State Department of Ecology Dam Safety Section, Initials "JML," Date May 30, 2000.

"Subsurface Conditions Data Report – West MSE Wall – Third Runway Embankment – Sea-Tac International Airport," Prepared for Port of Seattle and HNTB by Hart Crowser, June 2000.

"Preliminary Stability and Settlement Analyses – Subgrade Improvements – MSE Wall Support – Third Runway Project," Prepared for HNTB by Hart Crowser, June 2000.

I:\WR0380\SeaTac Comment Letter.doc

"Geotechnical Input to MSE Wall and Reinforced Slope Design – Third Runway Embankment," Hart Crowser Memorandum, August 21, 2000.

"Use of Advanced Testing Data – Sea-Tac Third Runway Project," Hart Crowser Memorandum, August 28, 2000.

"Port of Seattle – Sea-Tac International Airport – Reinforced Earth Design Calculations," Reinforced Earth Company, September 1, 2000.

"Subsurface Conditions Data Report – Additional Field Explorations and Advanced Testing – Third Runway Embankment – Sea-Tac International Airport," Prepared for HNTB by Hart Crowser, September 5, 2000.

"Methods and Results of Liquefaction Analyses – Third Runway Embankment – Sea-Tac, Washington," Hart Crowser Draft Memorandum, September 7, 2000.

"Stability Review of RECo 30% Design – Third Runway Embankment Project," Hart Crowser Memorandum, November 9, 2000.

"Seattle-Tacoma International Airport – The Journey Begins Here – The Third Runway," Presentation by Hart Crowser to the Technical Review Board, November 16-17, 2000.

"Proposed MSE Wall Subgrade Improvements – Seattle-Tacoma International Airport," Hart Crowser Memorandum, December 8, 2000.

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.

D

Response to Comments

Master Plan Update Improvements at
Seattle-Tacoma International Airport

Permit: 1996-4-02325

Prepared by:

Port of Seattle
17900 International Boulevard, Suite 402
SeaTac, Washington 98188

April 2001

GeoSyntec Consultants, February 16, 2001 letter

The responses in this section have been prepared from the Port's perspective and knowledge. In summary, the Port notes the following:

- Design of the walls is being done in accordance with accepted and proven procedures that are embodied in a nationally recognized building code;
- Because of the size and importance of this project, the Port has completed extensive exploration, testing and analyses, beyond that accomplished for most projects, and the design process is still ongoing;
- Performance of properly designed and constructed mechanically stabilized earth (MSE) walls in major earthquakes has been excellent. Based on this experience and incorporation of techniques used elsewhere that have withstood actual seismic challenges, the Port anticipates that the proposed MSE wall would withstand reasonable challenges;
- The Port has incorporated independent checks at every significant step in the process, including involvement of a highly qualified Engineering Technical Review Board.

Each of GeoSyntec's comments is specifically addressed below.

1A. Structural Integrity of the MSE Wall Foundation

Support for the mechanically stabilized earth (MSE) wall foundations will be dense and unyielding. The proposed use of "stone columns" is a form of subgrade improvement that will result in construction of a structural fill *in situ*. Use of the stone column technique provides a very adequate foundation that provides an alternative to making an open excavation immediately adjacent to Miller Creek and associated wetlands. This construction method avoids any potential short-term impacts associated with temporary construction dewatering.

Stone column construction is typically used to mitigate soils subject to seismic liquefaction, and/or to improve strength and reduce compressibility of native soils. This type of subgrade improvement is a widely accepted construction practice that has been used on major projects all over the world.

Stone columns are constructed by replacing soft or weak native soils with densely compacted angular rock that has much higher shear strength and bearing capacity than the original soils. The technique is discussed in detail in Appendix L of the Port's *Comprehensive Storm Water Management Plan*.

Stone column construction is well suited to verification of quality assurance during construction, and plans for such quality control verification are included in the current Phase 4 construction documents that have been available for review during the current §404/401 public comment period. The Port notes that Ecology and the Corps did not receive <u>any</u> comments critical of the proposed construction quality control and verification process for stone column construction.

The Port believes that the comment also suggests that design of the MSE walls is based on "limited" site-specific data. Actually design of the proposed MSE walls is based on more than 90 subsurface borings, cone penetrometer soundings and test pits, as well as an extensive series of *in situ* and laboratory soils tests. The exploration and test program generally conforms to standards for design of MSE walls published by the Federal Highway Administration (*Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines, SA-96-071, FHWA, 1997*) and the code developed for design of MSE walls by the American Association of State Highway and Transportation Officials

(AASHTO, 1996-2000 "Standard Specifications for Highway Bridges", 16th Edition, 1996, with current interim addenda through 2000).

1B. The Size of the MSE Wall is Accurately Reported.

Typical practice for mechanically stabilized earth (MSE) walls and all other types of structure, is to define their height above ground, i.e. the height of the MSE wall is typically measured from the toe to the top of the wall face. It is commonplace to design MSE walls that have a sloping ground surface above and behind the top of the wall face. As recommended in the design guidelines established by American Association of State Highway and Transportation Officials (AASHTO, *1996-2000 "Standard Specifications for Highway Bridges", 16th Edition, 1996, with current interim addenda through 2000*), the sloping ground behind the MSE wall is designed as a surcharge load to the wall and the slope below the toe of the MSE wall is designed as the wall embedment. The weight of the additional earth from the slope above the MSE wall has been taken into account as a surcharge load as recommended by AASHTO.

The MSE walls proposed by the Port range in maximum height from 50 to 135 feet. The firm designing these walls, RECo USA, has designed two MSE walls that were built to about the same height as the maximum proposed wall height at SeaTac: 137 feet high in South Africa and 133 feet high in Hong Kong. While neither of these two high walls had slopes above them, RECo has completed many such walls, including those listed below.

There are many tall MSE walls that have been successfully constructed with the sloping ground above the wall. Some examples are provided in the following table as a comparison to the Port's design. The first two of the examples, Le Peyronnet AB and Setouchi Country Club, are located in seismically active regions and have a total height (wall and slope on top) that is greater than the Port's design. Therefore, the Port's design is not unprecedented height for a wall with a slope on top.
Examples of MSE walls with sloping fill on top of the wall:

Country	Project	Combined Height of Exposed Wall and Slope on Top (feet)
Japan	Setouchi Country Club	240
France	Le Peyronnet AB	157
USA	**Proposed SeaTac Third Runway**	**153**
USA	US23, Tennessee	122
Mexico	Porta Del Sol	104
Japan	Highway Route 432	102

Source: RECo, March 2001.

The Port agrees with GeoSyntec that the proposed MSE walls are significant structures, and is providing the utmost level of care and attention to detail in the design.

2. The Port has Conducted Sufficient Laboratory Testing of Soils

Frequency of sampling and testing depends on variability of the soils and tests results, and with the level of experience of the engineer with the particular soils. Standard industry practice requires the design engineer to exercise professional judgment in determining the scope of exploration program and the frequency of sampling and testing based on examination of variability of ground conditions and test results. In the case of the Third Runway, the designers located the spacing of explorations to obtain samples for characterization of soil conditions and testing to generally conform to recommended FHWA practice (*Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines, SA-96-071, FHWA, 1997*).

Results of laboratory consolidated undrained (CU) triaxial tests on samples below the proposed West MSE Wall are consistent with results of strength tests from samples on other parts of the project. The laboratory strength test results also correlate well with the results of *in-situ* (field) cone penetration tests (CPT). It is the professional opinion of the Port's design team that the level and frequency of laboratory testing is appropriate based on the consistent results observed throughout the entire project site.

The Port's design team has taken a conservative approach in selecting design strength values of soils from results of both the laboratory and field tests. The shear strength values selected for the external or global stability analysis and design of the MSE walls are typically lower than those interpreted from laboratory test results. For examples, laboratory CU triaxial tests on fine-grained soils indicated that the value of effective friction angles ranged from 32 to 35 degrees, however, an effective friction angle of 32 degrees was used for the initial design analyses, and this was further reduced to 30 degrees in the latest stability verification analyses.

3. The Port has Accurately Interpreted Laboratory Strength Test Results

All the laboratory consolidated undrained (CU) and unconsolidated undrained (UU) triaxial tests were performed in accordance with the American Society for Testing and Materials (ASTM) standard procedures. The Port's design team used the procedures ASTM D 2850 "*Standard Test Method for Unconsolidated Undrained Compressive Strength of Cohesive Soils in Triaxial Compression*" to determine UU strength; and ASTM D 4767 "*Standard Test Method for Consolidated Undrained Compressive Strength of Cohesive Soils in Triaxial Compression*" to determine CU strength properties.

The test procedures in both ASTM D 2850 and ASTM D-4767 state that "the test load shall continue to a minimum of 15% strain, except loading may be stopped when the deviator stress has dropped 20% or when 5% additional axial strain occurs after a peak in deviator stress." All laboratory triaxial tests accomplished for the Third Runway project were terminated at 15% to 20% strain, as required by the ASTM standards.

The stress path plots in the CU triaxial test results showed essentially no difference in determining the effective friction angle of soils at 10% to 20% strain, since the stress paths converged on the same envelope prior to reaching the 10% strain level.

A close examination of the stress-strain curves in both the CU and UU triaxial tests indicates that 14 of the 37 soil samples (about 38%) showed higher shear strength at 20% strain than at 10% strain. The other soil samples showed either the same or slightly lower shear strength values at 20% strain compared to 10% strain. The difference in shear strength values at 10% and 20% strain is generally less than 15% and has already been taken into account in the Port's design. Running the tests to 20% strain demonstrates there is no significant reduction in strength as strain increases. This demonstrates the soil can tolerate large deformations without failure and any increase in strength means it will further limit deformations.

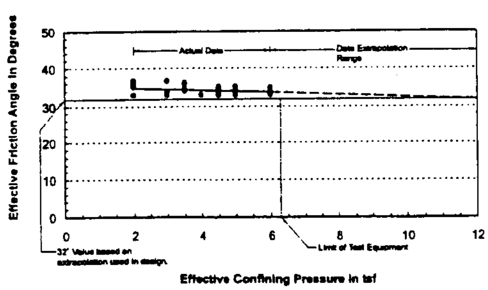
The design strength values of soils were selected based on the laboratory test results, as well as consideration of the field cone penetration test (CPT) data. The undrained shear strength of soils interpreted from UU triaxial tests correlates reasonably well with CPT results (*Kulhawy, F.H. and Mayne, P.W. (1990), Soil Property Manual, Electrical Power Research Institute, EPRI Report EL-6800)*. The selected design strength values of soils for the stability analysis and design of the MSE walls were typically lower (more conservative) than those interpreted from laboratory and field test results. For example, values of undrained shear strength used in the West Wall stability analyses were 1,000 pounds per square foot (psf) for the soft to medium stiff silt and clay, and 3,500 psf for the stiff to hard silt and clay, while actual UU strength values from samples at the West Wall location ranged from over 1,300 to almost 9,300 psf.

4. The Port has Employed Conservative Strength Values in Its Stability Analyses

The Port's design team agrees that the confining pressure used in the preliminary triaxial tests (about 6 tons per square foot, tsf) is less than the condition that will be produced by the maximum embankment height (up to about 11 tsf), but notes the range of confining pressures used represents the height range for much of the embankment. Higher pressures were not used in the preliminary triaxial tests because of a limitation in the capacity of testing equipment, but will be completed as part of final design.

The Port's design team used soil strength values that are reasonable and appropriate. The Port's site-specific triaxial CU test data produced effective friction values that ranged from 32 to 35 degrees and show a slightly decreasing trend as the confining pressure increases. Design analyses are based on the extrapolation of available test data to about 12 tsf, which produced an average effective friction angle for fine-grained soils of approximately 32 degrees. See Figure 1. The Port used 32 degrees as the basis for design in its global stability analyses. Moreover, subsequent analyses demonstrated factors of safety greater than 1.0 would result from using even lower values. Thus, the current design provides an additional margin of safety due to the use of this conservative angle of friction.

Sea-Tac Third Runway CU Triaxial Data



Note: Stability analyses show design is safe even if effective friction angle is reduced to 30 degrees.

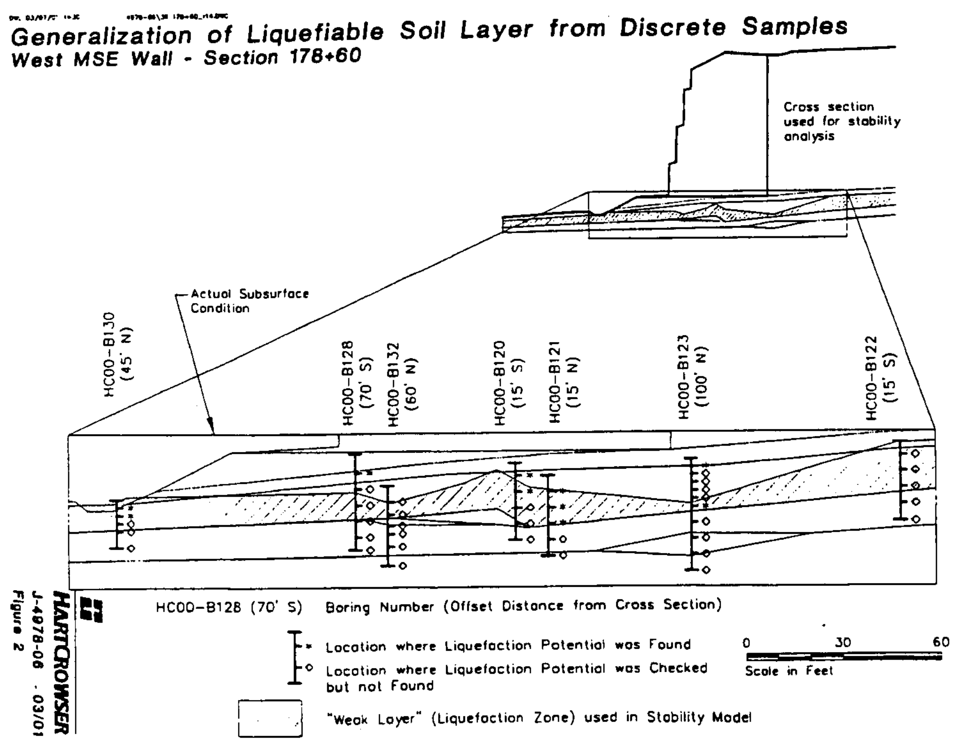
HARTCROWSER
J-4978-06 3/01
Figure 1

In addition to the checks described above, the Port's designers also noted that the effective friction angle of fine-grained soils interpreted from laboratory triaxial tests correlates well with field test (CPT) data *(Lunne, T., Christoffersen, H.P., and Tjelta, T.I. (1985). Engineering Use of Piezocone data in North Sea Clays. Proceedings, 11th ICSMFE, San Francisco, Vol. 2, pp. 907-912; and Senneset, K., Janbu, N., and Svano, G. (1982). Strength and Deformation Parameters from Cone Penetration Tests. Proceedings, Second European Symposium of Penetration Testing, Amsterdam, pp. 863-869).*

5. The Port's Liquefaction Analysis Methodology Is Accurate and Supported by the Scientific Literature.

A spatial analysis of potential liquefaction was completed along with a simulated spatial analysis based on a Monte Carlo type approach (*Hart Crowser, 2001. DRAFT Geotechnical Engineering Analyses and Recommendations, Third Runway Embankment, Seattle-Tacoma International Airport, SeaTac, WA. Pages 8 through 10, and A-6 through A-12, March 2001*). In some areas, the Port's consultant (Hart Crowser) did find specific seams or zones of potentially liquefiable soils; in other areas there are only discrete, isolated samples that analysis indicated are subject to liquefaction, and in these areas Hart Crowser found no geologic basis for interpolating contiguous liquefiable conditions. Analyses using the most conservative interpretation showed stability exceeded the target factor of safety.

Numerous cross sections for both MSE walls and the embankment were analyzed for stability based on conservative assumptions, using "weak seams" to represent continuous layers of liquefaction-susceptible soils. In several cases the Port's design analyses generalized liquefiable soils to be more extensive than actually exist in order to evaluate the effect on stability and to design the extent of subgrade improvement, see Figure 2 for example. Figure 2 shows how the Port conservatively modeled a few liquefiable samples as a continuous layer, for stability analysis.



In addition to stability analysis based on graphical interpolation and extrapolation of liquefiable soils, the Port's geotechnical engineer considered liquefaction in a statistical manner, to compare general trends in liquefaction potential based on four general subdivisions (North MSE Wall, 2H:1V Slope, West MSE Wall, and South MSE Wall). This comparison included considering the relative distribution of soils that would liquefy due to different size earthquakes, and what the resulting effect would be on soil strength.

It is the Port's belief that the commentor did not accurately address the screening criteria used by the Port to identify non-liquefiable soils, and the Port's analysis has not incorrectly applied screening criteria to identify liquefaction susceptible soils. The appropriateness of the Port's analyses is confirmed in the geotechnical engineering literature (*Seed, H.B., I.M. Idriss, and I. Arango, 1983. "Evaluation of liquefaction potential using field performance data," Journal of Geotechnical Engineering, ASCE, Vol. 109, No. 3, pp. 458-482; and Perlea, V.G., 2000. "Liquefaction of Cohesive Soils," Soil Dynamic and Liquefaction 2000 Geotechnical Special Publication No. 107, pp. 58-76).*

When referring to soils that do not meet all the screening criteria, Seed et al. (1983) specifically states that: "Otherwise clayey soils may be considered non-vulnerable to liquefaction." The Port's geotechnical consultant (Hart Crowser) used this method when they reported that: "if any one of these criteria was not met, the soil was deemed non-liquefiable." The commentor's assertion that "these criteria were developed for evaluation of materials that are potentially liquefiable, not identification of materials that are not liquefiable" is not supported by the literature on the subject. It is clear from the literature that the criteria can be used to exclude as well as include liquefiable soils.

The liquefaction susceptibility of soils with high fines contents were evaluated using the so-called "Chinese" criteria originally developed by Wang in 1979 (see *Wang, W., 1979. "Some Findings in Soil Liquefaction". Water Conservancy and Hydroelectric Power Scientific Research Institute, Beijing, China*); and later modified for consistency with U.S. practice by the U.S. Army Corps of Engineers (*Finn, W.D.L., Ledbetter, R.H., and Wu, G., 1994. Liquefaction in silty soils: Design and analysis. Ground Failures under Seismic Conditions, Geotechnical Special Publication 44, ASCE, New York, pp. 51-76.).* The Chinese criteria state that soils, which satisfy all of the four following soil conditions are susceptible to liquefaction:

Fraction finer than 0.005 mm $\leq$ 15%
Liquid limit $\leq$ 35%
Natural water content $\geq$ 0.9LL
Liquidity index $\leq$ 0.75

If liquefaction susceptibility requires the satisfaction of all four of these conditions, the lack of any on condition renders the soil non susceptible to liquefaction.

Additionally, the first of the four criteria above does not refer to "fines content" as assumed by the commentor. The comment uses the term "fines content" to refer to the "fraction of finer than 0.005 mm" criteria. The definition of "fines content" may be found in any soil mechanics text, or in ASTM D 653, which defines "fines" as the "portion of a soil finer than a No. 200 (0.075 mm) U.S. standard sieve." There is a tremendous difference in the dynamic behavior of soils finer than 0.075 mm and 0.005 mm.

Finally, the liquefaction analysis does predict liquefaction of soils with fines content of up to 100 percent, provided the screening criteria are met.

6. The Residual Shear Strength Values Used by the Port's Design Team Are Appropriate.

The preliminary analyses of the post-liquefaction residual strength prepared by the Port's consultant (Hart Crowser) were based on the mid-range of the empirical relationship developed by Seed and Harder (*Seed, R.B. and Harder, L.F. "SPT-based analysis of cyclic pore pressure generation and undrained residual strength," in J.M. Duncan ed., Proceedings, H. Bolton Seed Memorial Symposium, University of California, Berkeley, Vol. 2, pp. 351-376.* 1990). The empirical relationship developed by Seed and Harder represents the range of conditions where liquefaction has been observed. The mid-range of the empirical relationship was used to provide an estimate of the soil strength for analysis of stability under liquefaction conditions. The Port's final analyses, however, is based on the relationship developed by Idriss (*Idriss, I.M. Evaluation of Liquefaction, Potential Consequences and Mitigation, An Update. Presented at Vancouver Geotechnical Society, Vancouver, B.C., February 17, 1998*). This curve typically lies between the average and lower fifth of the range developed by Seed and Harder (which is comparable to the quartile or lower third range proposed by the commenter).

Extrapolation of the Seed and Harder data beyond the range of N =16 to 20 is common practice. In stating that extrapolation of residual strength to values above 600 psf represents "a dangerous design step without any theoretical or experimental evidence supporting their interpretation," the commentor is ignoring basic principles of soil mechanics and a large body of experimental evidence on the residual strength of liquefied soil. Laboratory test data extending back to the 1930s has established that the ultimate (large-strain) shearing resistance of soils increases with increasing soil density. There is a well recognized, unique relationship between large-strain undrained strength and density, a relationship later formalized as the steady state concept (*Castro, G., 1969. Liquefaction of Sands, Harvard Soil Mechanics Series 87, Harvard University, Cambridge, Massachusetts*). Extensive laboratory testing by a variety of researchers in the U.S. and abroad has shown that the steady state, or residual, strength of laboratory test specimens increases smoothly and continuously with increasing soil density. Because the standard penetration test (SPT) resistance of a given soil is also known to increase smoothly and continuously with increasing density of that soil, residual strength must also increase smoothly and continuously with increasing SPT resistance, as inferred by the original analyses (refer to *Gibbs, H.J., and W.G. Holtz, 1957. Research on Determining the Density of Sands by Spoon Penetration Testing, Proc. 4th Inter. Conf. Soil Mech. Found. Eng. (Zurich), Vol. 1, p. 126.; and Kulhawy, Fred H., and Paul W. Mayne, 1990. Manual on Estimating Soil Properties for Foundation Design. EL-6800 Research Project 1493-6, Electric Power Research Institute, Palo Alto, California*). The commentor correctly states that the Seed-Harder database does not contain observed residual strengths greater than 600 psf; it is also true that the database does not contain residual strength data for SPT resistances greater than 15. The reason for this limitation is quite simple – there are no documented cases of liquefaction flow failure in sandy soils with SPT resistances greater than 15.

The corrected soil N-value $(N_1)_{60}$ increases because the denser sol is more likely to dilate if deformed, thus exhibiting a much higher strength. However, the maximum strength that any location would be limited to the drained shear strength of the soil. Experience has shown that $(N_1)_{60}$ values greater than about 12 to 16 are invariably dilative, *and there are no documented cases of liquefaction flow in sandy soils with SPT resistances greater than 15.*

In addition to the original design analysis, which included the extrapolation described above, the Port repeated the analysis without the extrapolation, as a check during subsequent more specific analyses. In this check, the Port's design team limited residual strength to less than or equal to that predicted forsoils with blow counts of 16 (the limit of the Seed and Harder data) using Idriss' curve (Idriss, 1998) and re-analyzed stability using the re-calculated post-liquefaction residual strength. For this check, the Port found that the factors of safety in these stability analyses were greater than 1.1 except in one portion of the 2:1 embankment (near runway Station 206+44)) where the FS was 1.01. The Port has planned for subgrade improvement in that area.+

7. The Port Utilized the Correct Methodology for Pseudo-Static Analyses

The comment asserts that the Port's pseudo-static (seismic) stability analysis is improper, and that a more "proper" analysis should be performed to search for the critical failure surface independently of the static analysis. However, it is the Port's belief that there is no theoretical justification, or code requirement that justifies the suggested approach. The pseudo-static approach used by the Port represents the standard of practice for this type of analysis. Searching for a critical surface with the pseudo-static acceleration component included in the search is unreasonably overly-conservative, and for this reason is not required by design standards such as the code developed for design of MSE walls by the American Association of State Highway and Transportation Officials (*AASHTO, 1996-2000 "Standard Specifications for Highway Bridges", 16th Edition, 1996, with current interim addenda through 2000)* and the Federal Highway Administration (*FHWA, 1997, Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines, SA-96-071).*

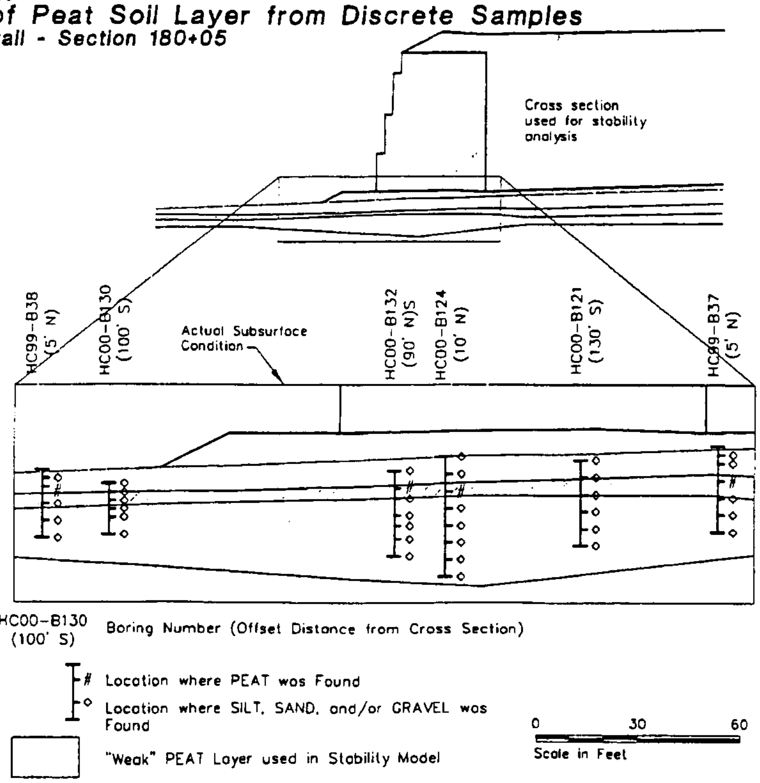
The Port recognizes that there are inherent limitations in the use of any pseudo-static, limit equilibrium type analysis to assess stability of slopes and MSE walls. The Port's engineers have addressed seismic stability recognizing the limitations in the pseudo-static method through the use of appropriate design parameters and factors of safety; use of post-liquefaction stability analyses, and in part by using a completely different approach (finite difference based deformation analysis) to provide an independent assessment of seismic stability.

The comment goes on to say that "sliding block" type failure surfaces should be considered in the analysis. The Port's design team did utilize sliding block or irregular surface analyses, (as described in the reports: *Hart Crowser, 2000. "Preliminary Stability and Settlement Analyses, Subgrade Improvements, MSE Wall Support, Third Runway Project", Appendix A June 2000*; and *Hart Crowser, 2000. "Stability Review of RECo 30 % Design - Third Runway Project," Hart Crowser Memorandum, November 9, 2000,* (i.e. analysis attachment pages 3, 6, 10 A & B, 11, 15, 17, 20, 28, and 40 through 42). The reported factors of safety for design include both circular and sliding block (or irregular wedge) type potential failure surfaces.

Not only did the Port's analyses include analysis of the sliding block type failure mode, many of its analyses included an artificially extended weak seam to verify that such a layer would not cause instability. This type of generalization is illustrated in enclosed Figure 2 (previously discussed) and Figure 3. Figure 3 shows an example of how intermittent isolated zones of peat were conservatively generalized into a weak layer, for purposes of the stability analysis.

The proposed subgrade improvement zone below each MSE walls was designed to provide a stable buttress assuming that there could be some zones of liquefaction or other weak soils below the embankment that are outside the zone of subgrade improvement. The enclosed Figures 2 and 3 illustrate specifically how the Port's analysis considered the potential effect of weak layers (liquefaction-susceptible soils and peat respectively) extending beyond the limits of the modeled cross-section. Since the proposed subgrade improvement zones were sized to provide a stable buttress to the embankment under both static and seismic conditions, there is no threat of weak soils below the embankment causing instability of the MSE walls.

Generalization of Peat Soil Layer from Discrete Samples
West MSE Wall Detail - Section 180+05



HARTCROWSER
J-4978-08 03/01
Figure 3

GeoSyntec states that "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 Port believes that this statement is correct when the soil stratigraphy allows the failure mass to increase in two dimensions, i.e. to extend to greater depths as well as farther back into the slope. However, that is not the case here, as the very strong glacial till provides a lower boundary to realistic potential failure surfaces. Indeed, the hypothetical critical surface drawn by GeoSyntec on Figure 1 of their review report shows a potential failure surface that extends only in the horizontal dimension (i.e. back into the slope but not deeper). It is relatively easy to show that the pseudo-static factor of safety *increases* when a pseudo-static failure surface of the type indicated by GeoSyntec extends further back into a given frictional soil.

As previously noted, the continuous peat layer shown in the illustration included in GeoSyntec's review comment does not actually exist, but was assumed as part of a "worst case" type analysis. Even if this surface did exist, GeoSyntec's conclusion that the critical pseudo-static failure surface would extend farther back would extend through the peat would only be accurate in the event that the pseudo-static analysis was performed incorrectly. Because the peat layer is relatively soft, upward propagating seismic waves refracted into the peat would, due to the low impedance ratio, have reduced stress amplitudes and therefore transmit lower driving forces into the potential failure mass. Use of the same pseudo-static coefficient for the entire potential failure mass would be incorrect.

8. The Probabilistic Seismic Hazard Analysis (PSHA) is Consistent with Standard Industry Practices

The comment expresses concern that the seismic environment of the project site has not been properly characterized, due to apparent inconsistencies in the PSHA. It is the Port's belief that the inconsistencies asserted to exist are not within the PSHA itself, but represent different assumptions used in the PSHA vs. the liquefaction analysis.

The commentor states "that the Hart Crowser acceleration response spectra (curves derived from the PSHA) agree remarkably well with the USGS values," and the Port believes that this is correct. The Port also believes that the earthquake magnitudes assigned to various recurrence intervals as part of the analysis of potential liquefaction are not completely consistent with the referenced USGS publication. It is the Port's belief that the magnitudes used in the Port's liquefaction analyses are more conservative than the referenced USGS publication.

For the liquefaction analysis only, the Port consultant assigned earthquake magnitude values that increased for longer recurrence intervals. This is a conservative way to account for the trend that increasingly larger magnitude earthquakes produce motions of longer duration. Hart Crowser is aware that a lower magnitude, local, shallow source, such as the Seattle Fault, could produce an equally high acceleration at the site as a higher magnitude subduction zone source further away. This assumption is limited to the analysis of potential liquefaction only, and not part of the PSHA. The Port's PSHA did not limit consideration of progressively larger events to the subduction zone.

The conservative assumptions in the liquefaction analysis are not interchangeable with the results from the PSHA (compare page 4 in *Hart Crowser, 2000, "Draft Memorandum: Revised Methods and Results of Liquefaction Analyses, Third Runway Embankment, Sea-Tac International Airport," with pages 1 through 10 and Figures 3 and 5 in Hart Crowser, 2001 "Additional Information on the Seismic Design, Sea-Tac International Airport", Memorandum to Embankment Technical Review Board, January 25, 2000.*

9. Three Time Histories are Being Used on the MSE Project

The commentor's criticism that the Port is using a single time history for this project presumably refers to a preliminary design memo (*Hart Crowser, 1999, "Sea-Tac Airport Third Runway, Probabilistic Seismic Hazard Analysis Results, Memorandum to Jim Thomson, HNTB", October 9, 1999*) and does not reflect the fact that three time histories are being used on this project, as recommended by the commentor. (For information on the two additional time histories, see *Hart Crowser, 2001 "Additional Information on the Seismic Design, Sea-Tac International Airport", Hart Crowser, January 25, 2000*).

The resonant frequency of the proposed MSE wall is not in the relatively "short frequency" (sic) range. The Port's analysis indicates the characteristic site period for the high wall (i.e., wall sections over 100-ft high) is on the order of 0.3 to 0.6 seconds, which corresponds to frequencies of 1.7 to 3.3 Hz. These are not particularly high frequencies. The design team believes the time histories used in the analyses are appropriate for the proposed construction and conditions at the site.

10. The MSE Wall Design Team Has Considered and Incorporated Seismic Performance Criteria into the Design.

The comment suggests that seismic ground motion criteria have not been developed for the project, and that the commentor could not identify established seismic performance criteria.

A number of different size earthquakes were evaluated as part of selecting the basis of design for the Third Runway MSE walls. Design is based on a level of ground motion with a return period of around 475 years. This value was developed using a probabilistic seismic hazard analysis (PSHA) that incorporates all relevant seismic sources and includes contributions from all earthquake magnitudes and distances from the site. As noted in the comment, this is the same criteria that was used by the Port for design of other major structures, including buildings that are occupied daily by thousands of air travel passengers and hundreds of Port employees. This basis of design is commonly used, and is appropriate, for structures occupied by humans or where failure could cause great harm.

The commentor disparages the 475-year criterion as the "Code requirement for ordinary buildings, e.g. for residential construction", and says this project is more important than typical residential construction. The Port disagrees, noting that the seismic standard used for the type of buildings where families reside, is an appropriate standard to use for design of these significant retaining walls.

It is important to clarify what an acceptable factor of safety for the 475-year criterion means in laymen's terms. The Port has designed the proposed MSE walls to meet various factors of safety for different conditions analyzed. Design for the 475-year event is based on satisfactory performance of the proposed walls, assuming the level of ground motion that has an average return period of 475 years. Further, the design team has sized the earth reinforcing components for the wall to allow it to handle these maximum earthquake loads after allowing for the level of corrosion that is expected for steel that has been buried in the ground for 50 years. Detailed deformation analysis for the maximum height MSE wall indicates maximum displacement for the wall is on the order of about one foot for this condition. This is anticipated to cause spalling of the concrete wall facing, but no failure of the reinforcing strips, -no catastrophic failure of the walls, and no displacement of the wall that would adversely affect Miller Creek, the integrity of the walls or functioning of the runway.

The Port's proposed design criteria for this project utilizes acceleration at this site which are much greater than the February 28, 2001 Nisqually Earthquake. While one may argue that another level of earthquake "should" be used, the simple fact is that the basis of design selected by the Port is the same as that used for many highway bridges and other major infrastructure. Seismic performance of MSE walls has been evaluated in a number of studies, both from a theoretical basis and after real earthquakes. See for instance: *Reinforced Earth Company, 1994, "Performance of the Reinforced Earth Structures Near the Epicenter of the Northridge Earthquake, January 17, 1994"; and Kobayashi, K. et al., 1996, "The Performance of Reinforced Earth Structures in the Vicinity of Kobe During the Great Hanshin Earthquake", International Symposium on Earth Reinforcement, Fukukoa, Kyushu, Japan, November 1996.* MSE technology is well established, and well-constructed walls of this type have performed well in seismic events.

Finally, the Port's MSE design is based on the methods specified by AASHTO, but the Port's design team has also included a number of provisions that go beyond AASHTO requirements. Standard approach to MSE design is based on limit equilibrium and ultimate strength type analyses. In addition to the Code requirements, the design analyses include stress-strain modeling to check and verify that deformations are within acceptable limits and that stresses in reinforcement do not exceed allowable limits.

11. Use of FLAC for Seismic Analysis is Well Documented in the Scientific Literature

This comment indicates a concern that the finite difference based computer code "FLAC" used by the Port has never been demonstrated to reliably predict seismic deformation of earth structures. Engineering literature in this area contradicts this contention and demonstrates the extensive use of FLAC for dynamic analysis of earth structures, including comparisons with real earthquakes. Examples of such literature, include:

Inel, S., W.H. Roth, and C. de Rubertis, 1993. "Nonlinear Dynamic Effective Stress Analysis of Two Case Histories," Proceedings of the Third International Conference on Case Histories in Geotechical Engineering pp 1735-1741.

Makdisi, F.I., Z-L Wang, and W.D. Edwards, 2000. "Seismic Stability of New Exchequer Dam and Gated Spillway Structure," Proceedings of the Twentieth Annual USCOLD Lecture Series: Dam O&M Issues - The Challenge of the 21st Century, pp. 437-458.

Bathurst, R.J. and K. Hatami, 1998. "Seismic Response Analysis of a Geosynthetic-Reinforced Soil Retaining Wall", Geosynthetics International, V. 5 Nos. 1-2, pp. 127-166.

Bathurst, R. J., and K. Hatami, 1999. "Earthquake Response Analysis of Reinforced-soil Walls Using FLAC," Proceedings of the International FLAC Symposium on Numerical Modeling in Geomechanics, pp. 407-415.

Roth, W.H., et al. 1993. "Upper San Fernando Dam 1971 Revisited". Annual Conference Proceedings of the Association of State Dam Safety Officials. D.W. Darnton and S.C. Plathby eds. Lexington, KY. pp. 49-60.

FLAC was used (or is being used) for Wickiup Dam in Oregon, Seymour Falls Dam in British Columbia, Rye Patch Dam in Nevada, and Pineview Dam in Utah. FLAC or similar procedures are being used to guide design of many earth structures, including both static and seismic analyses.

The Port's design team is very familiar with research at the University of Washington that includes use of FLAC for both static and seismic analyses of MSE wall performance (see for instance *Lee, W.F., 1997. "Numerical Analysis of Instrumentation of a Geosynthetic Reinforced Wall," Industrial Fabrics Association International: Geosynthetics, Vol. 1, pp. 323-336.).* The University of Washington research has demonstrated the reasonableness of FLAC analyses for seismic analysis of MSE walls based on comparison with shaking table and centrifuge test results.

Use of FLAC is above and beyond conventional design practice for MSE walls, i.e. the AASHTO Code that is being used by the Port. Use of this tool by the Port's design team provides an increased level of understanding regarding walls performance both during construction and service. The Port's design team selected FLAC as a tool to support the design process after considering capabilities of other dynamic modeling programs such as Plaxis and FLUSH. Use of FLAC enables the Port to estimate wall movement and stresses in the reinforcing for a wide range of conditions from construction through performance in various size earthquake events, a capability that is not equally available from alternative computer models.

The comment also included a number of technical questions that are addressed below:

- Default constitutive models & elements were used, based on demonstrated performance in FLAC models of MSE walls;
- Free- field boundaries were established such that their location did not affect the model;
- ProShake was used to calculate site response from bedrock motion to get input for base of model;
- Liquefaction deformation analysis was not accomplished in the FLAC analyses to date, but is being evaluated as a further check on wall performance
- The "composite strength" approach referred to in the comment was part of an analysis of part of the 2H: 1V embankment, and does not relate to design of the MSE walls. Shear strength of sand layers underlying the MSE walls was not simply weighted by the residual strength of liquefiable soils. Use of stone columns will mitigate potential for liquefaction in the areas where ground improvement is used. Strength of the soils in the subgrade improvement areas has been estimated using performance on other projects based on the area replacement ratio approach, and will be verified by testing during construction.

It is important to understand the fact that FLAC is only one of several tools/techniques used by the Port's design team to evaluate the seismic response of the MSE walls. It is also important to emphasize that the Port is not relying solely upon FLAC for the seismic design, but rather using it as an advanced tool to confirm and supplement the conclusions given by the more conventional analyses. The biggest benefit of FLAC is to help understand the mechanisms of deformation so that the reasonableness of the limit equilibrium analyses can be confirmed.

12. No Specific Source has Been Identified for Wall Backfill Material

The comment questioned why the Port has not provided test data from its own borrow sites to verify suitability for use as MSE backfill material. However, at this time, the identified borrow areas are not anticipated by the Port to be used as a source for MSE wall backfill materials.

Regardless of the source of the fill materials, the construction specifications will include provisions to test MSE wall backfill materials that are proposed for use by the Contractor. Such specifications are likely to be similar to specifications of the current Port of Seattle Phase 4 construction documents (which were available for review but were not addressed in these comments). MSE backfill material will, at a minimum, be tested as required to conform to the AASHTO Code being used for design, and to satisfy performance requirements discussed in *Hart Crowser, 2000. DRAFT Geotechnical Input into MSE Wall and Reinforced Slope Design, pages 5 through 12, August 21, 2000.* The fines content of the wall backfill will be limited to more stringent requirements than the Code, to provide improved drainage for the wall zone.

13. The HSA Techniques Were Appropriate and Did Not Lead To Erroneous Soil Characterization

This comment expressed concern that some of the drilling and sampling techniques used by the design team may not be appropriate and could produce errors in soil characterization. The Port's design team recognizes the issue raised in the comment but notes that any potential error of the type suggested would produce conservative results, i.e. it would <u>always</u> tend to make soils seem more susceptible to liquefaction than they actually are. Comparison of side-by-side cone penetrometer test (CPT) and SPT blow count (N) values for parts of the Third Runway project does indicate the N values are lower than might be expected, so it is likely that there would actually be somewhat less liquefaction due to the design earthquake than previously anticipated by the Port.

14. Construction Plans Should Include Instrumentation

The Port's design team agrees with this observation. Monitoring plans were discussed during scoping design for the MSE walls, and will be developed at the time final construction plans are prepared.

Monitoring during construction is an important aspect of geotechnical engineering that is very familiar to the Port's design team. The Port anticipates that the MSE monitoring plans will be developed by the wall designer (RECo), subject to review and concurrence by other members of the design team.

In general terms, construction monitoring is anticipated to include: 1) vertical deformation of the wall subgrade soils; 2) horizontal deformation of the wall subgrade soils; 3) horizontal deformation of the reinforced wall backfill; 4) horizontal and vertical movement of the wall face. Construction observations and monitoring data will be reviewed during construction to verify that the wall is performing in the manner anticipated by the designers. This type of monitoring is in addition to construction quality control tests and quality assurance procedures that will be incorporated into the wall & reinforcing component manufacture and field construction process.

15. Use of HELP Model Is Appropriate

The Port's design team understands the comment's concern about suitability of the HELP model for analysis of infiltration into landfills.

For the Third Runway project, HELP was used as part of a detailed hydrologic analysis that included several different models to analyze different aspects of the effect of the embankment on infiltration and groundwater recharge. The Port's approach used a model called Rosetta (*Schaap, M.G. and W. Bouten, 1996. "Modeling Water Retention Curves of Sandy Soils Using Neural Networks". Water Resour. Res. 32.3033-3040.*), that uses moisture-conductivity-suction relationships based on gradation of the fill materials, to develop parameter sets that control infiltration and unsaturated percolation into the embankment. The HELP model was used to simulate flow through different parts of the embankment, including the lateral drainage layer at the base of the embankment.

An Ecology consultant, Pacific Groundwater Group, used a different type of computer model and obtained results that are very close to results produced by the Port's analysis (*Pacific Groundwater Group, 2000. "Sea-Tac Runway Fill Hydrologic Studies Report", June 19, 2000.*)

16. Ecology Review of IWS Lagoon #3 Expansion

Ecology granted the Port a Dam Construction Permit on July 21st, 2000. In a letter to the Port, Ecology stated, "The approval is based on the fact that the plans and specifications are acceptable." Ecology also stated that periodic site visits would be conducted during construction to confirm work is progressing according to plan, but gave no indication of any other review or independent analysis. See also General Response GLR14.

17. There Will Be No Material Impact On Existing Contaminated Groundwater From the Construction of the Third Runway.

In the area of the Airport where most aircraft fueling and maintenance operations have been performed (called, for the Model Toxics Control Act Ground Water Study, the Airport Operations and Maintenance Area, AOMA) contaminated ground water exists in a number of localized, discrete sites. The horizontal boundaries of each contaminated ground water site are defined by site investigation data, and include any migration that might have occurred due to the presence of utility and underground infrastructure that crisscross the entire AOMA.

Within the AOMA, defined areas of contaminated ground water exist in both shallow perched zones and in the shallow regional aquifer (Qva). The perched zones are isolated and discontinuous, while the Qva is continuous, the uppermost aquifer of regional extent in the airport vicinity.

Underground infrastructure and utilities are typically, constructed at higher elevations than the location of the perched zones within the AOMA. Despite the numerous underground infrastructure and utilities that could influence perched ground water contamination in the AOMA, investigation data demonstrate that existing perched zone contamination has remained localized, i.e., has not migrated significantly along utility pathways, and remains well within the AOMA. Given this result, together with the discontinuous nature of the perched zone, the Port expects expect no material impact from the construction of Third Runway and other infrastructure on existing contaminated ground water in the perched zone.

Underground infrastructures are rarely constructed at depths where impact to the Qva is likely, but do exist (e.g. the satellite subway and baggage system tunnels). In one instance, AOMA Qva contamination migration has been impacted somewhat by the presence of deep infrastructure, but still remains localized and well within the AOMA. No such deep infrastructure is planned for the Third Runway. Some deeper infrastructure may be constructed for other Master Plan projects (e.g., STS upgrades or SASA), but these would be in locations far from known Qva ground water impacts. Therefore, the Port expects no material impact from the construction of Third Runway and other infrastructure on existing contaminated ground water in the Qva. In addition, construction within contaminated areas will include monitoring and remediation consistent with MTCA and other applicable environmental regulation. Such remediation may include the removal of contaminated soil to appropriate offsite treatment and disposal facilities.