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6 August 2001

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Subject: Comments on Recently Received Documents Pertaining to Seattle Tacoma International Airport Project Third Runway – Embankment Fill and West MSE Wall

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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. GeoSyntec first commented on project documents in a letter dated 16 February 2001. This letter summarizes GeoSyntec's comments on documents relating to the project which were not available during the first review. The new documents reviewed are the following:

- "Geotechnical Engineering Analyses and Recommendations – Third Runway Embankment – Seattle-Tacoma International Airport – SeaTac, Washington," Prepared for HNTB by Hart Crowser, December 4, 2000.

- "Additional Information on the Seismic Design," Hart Crowser Memorandum, January 25, 2001.

- "Revised Methods and Results of Liquefaction Analyses – Third Runway Embankment – SeaTac, Washington," Hart Crowser Memorandum, March 5, 2001.

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Exhibit 405
Date 2/19/02
Witness Karazanjian
Diane Mills Court Reporter

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GeoSyntec's personnel in charge of the review include Patrick C. Lucia, Ph.D., P.E., and Edward Kavazanjian, Jr., Ph.D., P.E.

Introduction

In earlier letters dated 16 February 2001, 15 March 2001, and 22 June 2001, GeoSyntec has identified consistent gaps in the analyses carried out by the Port of Seattle and its consultants in their efforts to design this project. The review of additional documents presented herein unfortunately follows this trend. A project of the magnitude and importance of the Third Runway Expansion at the Seattle Tacoma International Airport requires adherence to the highest of standards. The various elements of the design process must be scrutinized at every level in order to give the resulting structure every chance of weathering extreme events and remaining safe and operational for the economic life of the facility and beyond.

In the discussion that follows, several key points and concerns will be made regarding the analyses presented in these recently reviewed documents. In our discussions, we focus on the following essential findings:

- the seismic risk presented by the numerous sources of potential earthquakes in the Seattle region have not been addressed comprehensively and has been underestimated for one or more of the major sources;
- seismic stability analyses and FLAC deformation analyses have been performed incorrectly;
- the extent of the potentially liquefiable material was not properly represented, resulting in overestimation of post-earthquake stability;
- composite shear strengths used to represent the weak liquefiable materials are being overestimated, resulting in overestimation of post-earthquake stability; and
- insufficient documentation is being provided for a proper and thorough regulatory decision to be made.

We continue to maintain that the Port's consultants have not demonstrated that this project as designed will perform acceptably when subjected to the seismic loads that are anticipated over its useful life. The following sections provide our commentary on each of the three documents reviewed, focusing on the above issues.

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Document 1: *"Geotechnical Engineering Analyses and Recommendations – Third Runway Embankment – Seattle-Tacoma International Airport – SeaTac, Washington," Prepared for HNTB by Hart Crowser, December 4, 2000*

Comment 1A: <u>By using composite strengths as implemented, the Hart Crowser analysis may overlook a potentially dangerous slope instability mechanism.</u>

Based upon a statistical assumption that one-third of the soil would liquefy in the design event, Hart Crowser uses the following equation to define the post-earthquake composite strength of a soil layer which contains both liquefied and unliquefied soils for the 475-year earthquake event:

$$\tau = \frac{1}{3}c + \frac{2}{3}\sigma_v \tan(\phi)$$

where τ is the composite shear strength, c is the residual strength of the liquefied soil (the one-third represents the assumption that one-third of the soil would liquefy), σ_v is the stress caused by the overlying embankment and computed based upon the mid-height of the embankment, and ϕ is the friction angle of the unliquefied soil (the two-thirds represents the strength contribution of the soil that is assumed not to liquefy). By applying this uniform strength value throughout the entire soil layer, Hart Crowser may be seriously overestimating the strength at the toe of the slope where the height of the overlying embankment is very small, and therefore σ_v is very small. By increasing the strength at the toe, a potentially serious progressive yielding mechanism is being ignored.

Progressive yielding occurs when a slope failure does not occur all at once (the underlying assumption of limit equilibrium analyses), but rather a portion of the failure surface weakens first, and as a result more stress that was previously carried by this soil is transferred to other portions of the failure surface and they in turn begin to fail progressively due to the increased stresses. For the embankment, a loose liquefiable zone near the toe may lose strength due to the earthquake, followed by a progressive shearing of soils further underneath and up into the embankment. The process can perhaps be described as an unzipping of the failure surface, progressing from one end to the other.

Proper analysis of this mode of failure requires that the potentially liquefiable soil, or any other soil that is subject to strength loss over the project lifetime due to seismicity or any other reason, be modeled in different segments, each with an appropriately selected strength value. Therefore, rather than using a single composite strength value, the soil below the toe of the embankment should be given a residual or nearly residual strength value, and higher strengths should be applied incrementally for materials under the mid-height of the embankment, and materials under the full embankment thickness.

This post-earthquake strength formulation should be employed both for the limit equilibrium stability assessment and for the FLAC deformation analyses. In fact, one of the primary advantages of using a numerical analysis tool like FLAC is that it allows the engineer to track progressive yielding.

Comment 1B: The composite strengths calculated in the report appear to be incorrect relative to the described methodology, resulting in unconservatively high strength values.

The methodology used for calculation of composite strength was discussed in the previous comment. The equation is repeated here:

$$\tau = \frac{1}{3}c + \frac{2}{3}\sigma_v \tan(\phi)$$

Using this methodology, Hart Crowser states in their report that the following results were obtained "by using one-half of the embankment height to calculate the vertical overburden stress":

- Height of 40 feet, $S_{avg} = 2{,}472$ psf
- Height of 65 feet, $S_{avg} = 3{,}885$ psf
- Height of 85 feet, $S_{avg} = 5{,}015$ psf
- Height of 115 feet (max for Station 193+19), $S_{avg} = 6{,}709$ psf

Based on our understanding of the stated intent of these strengths, the computed values are incorrect, and potentially very unconservative. The following table demonstrates the correct calculation using Hart Crowser's equation for composite strength:

c Residual Strength (lbs/ft²)	ϕ Frictional Strength (degrees)	Fill Unit Weight (lbs/ft³)	Embankment Height (ft)	One-Half Embankment Height (ft)	σ_v Vertical Overburden Stress (lbs/ft²)	τ or S_{avg} Composite Strength (lbs/ft²)
675	32	135	40	20	2700	1350
675	32	135	65	32.5	4388	2053
675	32	135	85	42.5	5738	2615
675	32	135	115	57.5	7763	3459

The composite strengths presented by Hart Crowser in their report are the result of using the full embankment height instead of the one-half embankment height (e.g., calculating overburden stress based on 115 ft instead of 57.5 ft of compacted fill). If the error has propagated throughout the analysis, then the strengths being used are overestimated by close to a factor of 2, potentially resulting in a highly unconservative analysis.

Additionally, the value of 675 psf used for the residual strength of the liquefied materials is based upon an earlier liquefaction analysis. Based on liquefaction analyses performed in the subsequent 5 March 2001 Memorandum titled "Revised Methods and Results of Liquefaction Analyses," the residual strength should range from approximately 400 to 600 psf, which would have the effect of additionally reducing the composite strengths by up to 100 psf.

Comment 1C: The random distribution of post liquefaction residual strengths in the FLAC analyses ignore basic geology and render the analysis ineffective.

The statistical implementation of liquefied residual strengths in the FLAC analyses are not based on an understanding of the geology of the site and do not form a basis for a sound design. The distribution of liquefiable soils throughout the site is controlled by depositional processes and is not likely to be random, but rather the systematic result of meandering stream channels or other such geological processes that left behind deposits of loose silts and sands. While the distribution of liquefiable deposits may be complex, it is not random and should not be modeled as such.

The soil model developed by Hart Crowser for use in FLAC analyses represents the potentially liquefiable soil layer as a grid of cells in which loose, liquefiable materials are disconnected from each other and surrounded by much stronger materials, thereby greatly reducing the model's ability to predict realistic deformations. Rather than the random scatter approach employed in the analysis, analyses at any given cross-section should be based upon the geological information present at that location. Where exploration data indicates loose material, a continuous weak seam should be assumed consistent with the geological process that created the loose deposit. This will most likely result in an increased level of deformation and/or reduced factor of safety.

Hart Crowser acknowledges this unconservatism in their analysis in their report, stating that: "It is important to note that if the zones of liquefaction are not randomly distributed (i.e., if there is a significant zone of liquefaction in a critical area, such as occurs north of Station 205+00) actual deformations would be greater than these calculated deformations for discontinuous liquefaction." Therefore, by their own admission, this use of a random distribution of liquefiable material is a significant issue with respect to the conservatism, or lack thereof, in their analyses. Basic geological principles were ignored in the analysis and an admittedly unconservative approach to characterization of potentially liquefiable zones was employed in the analysis.

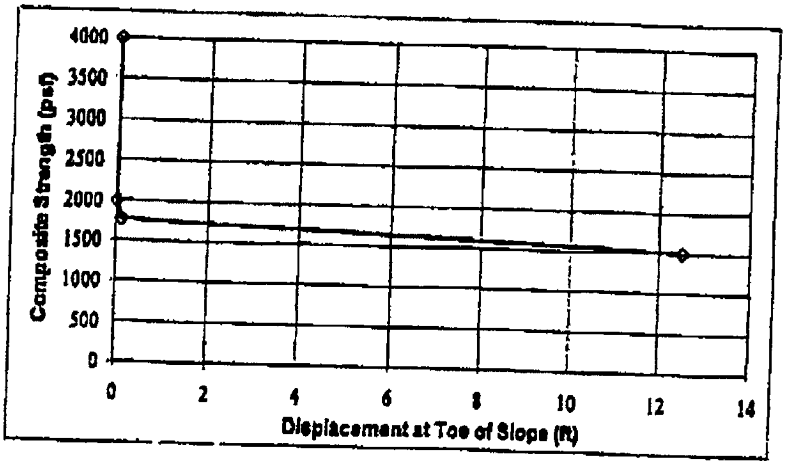
Comment 1D: Results of the FLAC deformation analysis are indicative of either numerical instability in the program or extreme sensitivity of the analysis to capacity and demand values rather than deformations that can be used as a basis for design.

Table A-8 of the report is presented here, showing results for FLAC deformation analyses using composite strengths:

Table A-8 - Displacements Calculated using the Composite Strength Method

Composite (Residual) Strength in Psf	Horizontal Displacement in Feet		
	toe of slope	midheight of slope	top of slope
4000	0.0	0.0	0.0
2000	0.0	0.1	0.1
1750	0.1	0.3	0.2
1500	12.5	15.1	9.0
1250	unstable	unstable	unstable

The displacements calculated at the toe of the slope are plotted in the following graph:



Hart Crowser states: "These results indicate that the composite strength needs to be approximately 1,750 psf or larger to keep displacements to a reasonable level (on the order of inches) during the design level seismic event."

Our interpretation of the results is that, as implemented, the FLAC analysis is encountering either numerical instabilities (i.e., significant mathematical errors are being generated as the program performs calculations) or physical instability. The evidence of this is the progression of deformation calculations. As the strength is decreased from 2000 psf to 1750 psf, an increase in deformation of 0.1 ft occurs at the toe of the slope. Then, an equivalent decrease in strength of 250 psf, from 1750 psf to 1500 psf, leads to an increase in deformation from 0.1 ft to 12.5 ft at the toe of the slope. The next decrease in strength by the same amount leads to an "unstable" output, which typically either means that the program could not compute an answer, or the answer it produced made no physical sense (e.g., it may have computed 100 ft of deformation).

Presumably, the same pattern of instability would be observed if the shear strength was held constant and the seismic load was gradually increased. This type of extreme sensitivity to seismic load or resistance is one of the driving factors behind the trend towards Performance Based Design (PBD), wherein seismic analysis can not look at a single design level, but must consider seismic performance over a spectrum of load levels, up to and including the collapse load. We note that Professor Steven Kramer of the University of Washington, a consultant to Hart Crowser on this project, is the geotechnical coordinator for the Pacific Earthquake Engineering Center (PEER), one of the champions of the PBD design methodology. PEER's stated mission is "to develop performance-based earthquake engineering technologies to meet the economic and safety needs of owners and society."

Insufficient information has been provided to assess the validity of the FLAC model used in the analysis (including the constitutive models employed for the various materials and the rationale behind selection of parameters). Any modeler using programs such as FLAC for complex analysis must be aware of the possibility for numerical instabilities in their analysis, must know how to recognize it when it is there, and must be able to judge the effects of such instabilities on the results. There are several other parameters besides strength that may produce equally large variations in results with only small variations in input. A proper modeling effort should include a parametric analysis aimed at evaluating the sensitivity of the outcomes to the input parameters. The results of the parametric analysis should be used to assess the appropriate range of input parameters. On a larger scale, the suitability of the selected soil constitutive model (i.e., the mathematical model that describes the behavior of the soil under different stress conditions) and the geometric model (i.e., the size and shape of the mesh used to represent the physical configuration of the different materials) must be evaluated in similar fashion.

The results in Table A-8, showing large variations in deformations as a result of very small changes in strength (i.e., 250 psf), are indicative of a FLAC model that is overly sensitive to the input parameters. Therefore, the results do not provide confidence in the outcomes and should not be used as a basis for design, especially given that variations in strength of 250 psf within a given soil deposit and proportional variations in seismic loading are commonplace.

Comment 1E: Instability in slopes up to 60 feet high should not be considered a "less significant problem" as it could have potentially serious effects or lead to further progressive slope failures.

Hart Crowser states: "Parametric analysis indicates that smaller scale instability involving slopes up to about 60 feet in height could result from local discontinuous zones of liquefaction. Depending on where liquefaction occurs and the magnitude of strength reduction, such events could limit use of the airfield perimeter road and require local slope maintenance." This issue is considered a "less significant problem" by Hart Crowser. While it may be a smaller scale problem than a slope failure through the West MSE Wall, a 60 ft slope stability concern is not an insignificant concern.

Depending on the location and type of failure, a 60 ft slope failure could lead to a larger subsequent failure, leading to more extensive damage and higher repair costs. Additionally, a 60 ft slope failure could impact not only the perimeter road, but could potentially impact wetlands adjacent to the slope. As the details of the parametric analyses referred to in the Hart Crowser statement are not provided in the report, insufficient information has been provided to assess the impact of these "less significant" slope failures.

Comment 1F: Additional Concerns

- In our previous review of project reports, we commented that Hart Crowser had not performed stability analyses looking at potential failure surfaces that travel along the weakest materials. This issue was discussed in previous letters by GeoSyntec (16 February 2001 - Comment 7; 22 June 2001 – Comment 7). As this report still does not provide documentation showing the failure surfaces analyzed under the residual strength conditions, we continue to have this very significant concern.

- Note (2) of Table A-2 states that "Undrained strength parameters were used for the end-of-construction cases, otherwise, drained strength properties were used." Were undrained strengths for these silt/clay materials also used for the seismic analysis, which should be considered an undrained event?

- Water levels used in the liquefaction susceptibility analysis are consistently referred to by Hart Crowser as being conservative, and yet they discuss the large uncertainties in the measured values. Typically, when water levels cannot be accurately determined, it is standard engineering practice to assume that they are as high as can be reasonably foreseen. If this practice is being implemented, it should not be considered as a source of conservatism in the analysis, especially considering the significant unknowns involved in the prediction of the effects of embankment construction on the future water elevations.

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Document 2: ***"Additional Information on the Seismic Design,"- Hart Crowser Memorandum, January 25, 2001***

Comment 2A: A 10% in 50 year design seismic event is insufficient for a critical lifeline facility such as the Seattle-Tacoma International Airport.

Based on results of the Probabilistic Seismic Hazard Analysis (PSHA) a 475 year seismic event, corresponding to a 10% probability of exceedance in 50 years, was selected by Hart Crowser as the basis for developing time histories for use in design. As stated in our 16 February 2001 letter (Comment 10), we do not believe that this is an appropriate design level event as the Seattle-Tacoma International Airport is an important lifeline structure that must remain in service after a significant earthquake. The Seattle region has historically been subjected to events of much greater magnitude than the proposed M_w 7-7.5 design event.

We do not believe analysis for a single probabilistic design level adequately characterizes the seismic performance of any facility. For a facility as important as the Third Runway, we recommend the use of performance-based design, where the adequacy of the proposed MSE Wall and embankments are evaluated based on their performance over the entire spectrum of feasible events up to the maximum credible event or the collapse load. As noted previously, the PEER Center earthquake engineering research consortium, including the University of Washington, have championed this approach to rational seismic design. Such an evaluation would provide a sound basis for making fully informed decisions on project issues incorporating safety concerns, environmental concerns, and economic concerns. As the airport is a key lifeline facility essential to both emergency response and recovery efforts following a major seismic event, at a minimum seismic performance of the MSE wall and embankment should be evaluated under conditions representative of the historical "great" earthquakes of the region.

Comment 2B: The hazard associated with a major Cascadia interplate event may be underpredicted.

The very large Cascadia interplate events (M_w 8.5-9) are represented in the seismic hazard analysis using a "characteristic earthquake" model. While this model considers the historical data on recurrence intervals for the earthquake, it appears to be implemented such that the recurrence of such an earthquake is random. In the hazard analysis, the Cascadia interplate event is represented by two characteristic earthquakes: 1) M_{max} = 8.5 to 8.7 with a recurrence interval of 411 years and 67% weighting; and 2) M_{max} = 8.8 to 9.0 with a recurrence interval of 983 years and a 33% weighting. The analysis does not appear to take into account the widely accepted evidence suggesting that the last Cascadia interplate event in the Seattle region occurred approximately 250 to 300 years ago. Incorporation of this elapsed time since the last event into the hazard analysis may significantly increase the probability of a major Cascadia interplate event in the near future (i.e., in the 50-year period considered in the analysis), particularly given the 67% weighting to the event with a 411 year recurrence interval. There is a significant probabilistic difference between a recurrence interval of 411 years with no knowledge of the last event and a recurrence interval of 411 years with the knowledge that there are only 100 to 150 years left to span the mean recurrence interval.

Comment 2C: The activity rate of the Cascadia subduction zone intraplate event has been underestimated.

The Gutenberg-Richter parameters selected to represent the recurrence of Cascadia intraplate events were a = 2.219 and b = 0.652. These numbers are used with the Gutenberg-Richter recurrence law as follows:

$$\log\lambda_m = a - bm$$

where λ_m is the mean annual rate of exceedance of an earthquake of magnitude m. In other words, λ_m is the inverse of the mean number of years between events of magnitude m (e.g., if $\lambda_m = 0.01$ then $1/\lambda_m = 100$ years). Incorporating the parameters a = 2.219 and b = 0.652 in the recurrence law yields recurrence rates of about 100 years and 220 years for events of magnitude 6.5 and 7 respectively.

However, the Cascadia intraplate source was the source for the 1949 M_w 7.1 Olympia event, the 1965 M_w 6.5 Seattle-Tacoma event, and the recent 2001 Nisqually event (M_w about 6.7). That corresponds to three events over magnitude 6.5 in 52 years, which is significantly greater than one event every 100 years as predicted by the recurrence law with the given parameters. This clearly indicates that the local activity for this source is higher than that which was incorporated in the PSHA by Hart Crowser.

Comment 2D: <u>Modeling of the shallow crustal sources with the Gutenberg-Richter recurrence law included numerous earthquakes with very low magnitudes, which can adversely affect the computed parameters and lead to an underestimation of the recurrence intervals at higher magnitudes.</u>

According to Hart Crowser, the database used to develop recurrence relationships "included all recorded events of magnitude greater than 2.0." Determination of the a and b parameters discussed in Comment 2C is highly sensitive to the minimum magnitude used in the database. According to Prof. Steven Kramer of the University of Washington, "In most PSHAs, the lower threshold magnitude is set at values from about 4.0 to 5.0 since magnitudes smaller than that seldom cause significant damage."[1]: The effect of including the low magnitude earthquakes is to skew the recurrence relationship defined by parameters a and b towards these insignificant earthquakes, thereby reducing the impact of the larger magnitude earthquakes that are of primary concern.

Comment 2E: <u>The synthetic time histories do not capture the contribution of the M_w 8.5-9 Cascadia interplate event, resulting in a dangerous lack of seismic energy in the long period range.</u>

Neither of the synthetic time histories presented capture the contribution to the seismic hazard of the M_w 8.5-9 Cascadia interplate event. This event may dominate the acceleration response spectra (ARS) at long periods (2 to 4 seconds). By not including a time history which is representative of this significant component of the seismic hazard, Hart Crowser is performing an incomplete, and therefore potentially dangerous, analysis. The suite of time histories used in the analysis should include a motion representative of a Cascadia interplate event. This motion may not match the peak ground acceleration (PGA) of the design event, but should be representative of the design ARS in the long period range and should have a longer significant duration than the two synthetic time histories cited in this report.

[1] Kramer, Steven L. (1996) *Geotechnical Earthquake Engineering*, Prentice Hall (quote from pg 123).

Comment 2F: The use of the bracketed duration based on Chang and Krinitszky (1977) to select representative time histories is inappropriate.

The "bracketed duration" was defined by Bolt (1973)[2] and used by Chang and Krinitszky (1977)[3] to develop a set of "typical" earthquake durations at soil sites. Hart Crowser used the typical soil bracketed durations recommended by Chang and Krinitszky as a basis for selection of appropriate seed motions to develop synthetic time histories for the site. The database used in the Chang and Krinitszky study is inadequate relative to the wealth of data that has been collected in the subsequent 24 years, and therefore should not be used for a project of this importance. Furthermore, the Chang and Krinitszky database has not been updated because the profession has moved away from use of the bracketed duration to characterize strong ground motions. Most current seismological studies use the significant duration of Trifunac and Brady (1975)[4], or some derivative thereof, to characterize the duration of strong ground motions.

Document 3: ***"Revised Methods and Results of Liquefaction Analyses – Third Runway Embankment – SeaTac, Washington," Hart Crowser Memorandum, March 5, 2001***

Comment 3A: Reductions in strength should be considered for soils with factors of safety against liquefaction greater than 1.0.

In discussing residual excess pore pressures which are generated in soil during an earthquake and remain in the soil for a period of time afterwards, Seed and Harder (1989)[5] have stated:

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[2] Bolt, B.A. (1973), "Duration of Strong Ground Motion," Proc. 5th World Conference on Earthquake Engineering, Rome, Italy.

[3] Chang, F.K. and Krinitszky, E.L. (1977) "Duration, spectral content, and predominant period of strong motion earthquake records from western United States," Miscellaneous Paper 5-73-1, U.S. Army Corps of Engineers, Waterways Experiment Station, Vicksburg, Mississippi.

[4] Trifunac, M.D. and Brady, A.G., (1975), "A Study of the Duration of Strong Earthquake Ground Motion," *Bulletin of the Seismological Society of America*, Vol. 65, pp. 581-626.

[5] Seed, R.B. and Harder L.F. (1990) "SPT-Based Analysis of Cyclic Pore Pressure Generation and Undrained Residual Strength" Proceedings of the H. Bolton Seed Memorial Symposium, Volume 2, BiTech Publishers Ltd.

... it appears that a reliable analysis can be performed by considering that:

1. *Soil elements with low factors of safety [against liquefaction] ($FS_l \leq$ 1.1) would achieve conditions wherein soil liquefaction failure should be considered to have been "triggered", and undrained residual strengths (S_r) should be assigned to these zones for further stability and deformation analyses.*

2. *Soil elements with a high factor of safety ($FS \geq 1.4$) would suffer relatively minor cyclic pore pressure generation, and should be assigned some large fraction of their static strength for further stability and deformation analyses.*

3. *Soil elements with intermediate factors of safety ($FS \approx 1.1$ to 1.4) should be assigned strength values somewhere between (though in some cases including) the values appropriate to conditions 1 and 2 above. Whether the values assigned should be nearer to the initial static strength or to the residual undrained strength is a function of FS_l, whether or not the soil is judged to be strongly contractive in unidirectional shearing (and thus potentially vulnerable to "progressive" failure), and levels of uncertainty involved in various steps of the analysis up to this point (for any specific case).*

In the analyses by Hart Crowser, it appears that reduced strengths are only considered for materials that have a factor of safety against liquefaction (FS_l) ≤ 1.0. This leaves the range of materials with FS_l between 1.0 and 1.4 at full strength, therefore ignoring potentially dangerous stability conditions resulting from an earthquake event.

Comment 3B: <u>The extent of potential liquefaction may have been underestimated, which could in turn lead to significant overestimation of the stability of the embankments and MSE walls.</u>

We continue to question the use of the Chinese Criterion for elimination of potentially liquefiable materials. As we have stated before, there are soils that are commonly found in the Seattle area (e.g., glacial soils with high "non-plastic" fines content) that may be susceptible to liquefaction, and yet would be identified as non-liquefiable according to this screening method. Failure to identify potentially liquefiable soils in the foundation materials is a potentially fatal flaw in the seismic stability assessment.

Comment 3C: Results of cone penetration testing (CPT) can provide valuable insight into the liquefaction susceptibility of a site, and yet this data was not included in the analysis.

Hart Crowser states: "Cone penetrometer tests accomplished by Hart Crowser and others supported interpolation of the standard penetration test (SPT) results, but are not included in the analysis presented in this draft memorandum."

Hart Crowser does not explain why the CPT data was not included in the database used for liquefaction analysis. CPTs can provide a significant amount of data and are both suitable and widely used for liquefaction analysis. CPT data is generally recognized as more repeatable and, in the absence of energy measurements made during SPT testing, more reliable than SPT data. Hart Crowser has stated on several occasions that they have attempted to expand the database of samples for the liquefaction analysis in order to improve the statistical validity of the outcomes, and yet they have not included the wealth of information that is typically provided by CPTs.

Conclusions

In a 3 May 2001 Memorandum for Record, Muffy Walker of the US Army Corps of Engineers (USACE) asks the following questions:

> "... does the information provided by the Port support their conclusions that the walls are viable, safe options and there will be no net adverse effects on stream flows and/or wetlands?"
>
> "... does the geological information provided support the conclusion regarding the types of subgrade improvements which need to be made and the seismic stability of the wall?"
>
> "under extreme earthquake conditions ... can movements [to the Wetland 37 Wall] cause a structural failure to the wall and allow fill material to slough out potentially causing impacts to wetlands and Miller/Walker Creeks?"

Through a series of previous letters as well as this one, GeoSyntec has identified persistent gaps in the analyses carried out by the Port of Seattle's consultants in their efforts to design this project. The gaps we have identified show that these questions have not been adequately addressed for either the walls or the embankments. We have raised numerous substantive questions about the design analyses that have not been satisfactorily answered by the Port's consultant. On behalf of the Airport Communities Coalition, we ask that prior to regulatory certification or approval of the proposed Third Runway Project the Port of Seattle be required to respond to the issues raised in these letters and that we be granted the opportunity to provide follow-up review and comment on that response.

Sincerely,

Patrick C. Lucia

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

for

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

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

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