Bozcboom
EXHIBIT NO. 243
2-5-02
M. Green

February 15, 2001

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Subject: Comments on stormwater, hydrology, and hydraulics aspects of proposed 3rd runway and related development actions at Seattle-Tacoma International Airport, Corps Reference No. 1996-4-02325.

Northwest Hydraulic Consultants has been retained on behalf of the Airport Communities Coalition to provide technical reviews of stormwater, hydrology, and hydraulics elements of proposed development actions at SeaTac airport. Our comments on the November 1999 version of the project stormwater management plan and related environmental documents were submitted to Ecology and the Corps in a series of three letters dated 11/24/99, 5/3/2000, and 7/31/2000. Our comments on the August 2000 version of the stormwater management plan were submitted to Ecology (but not the Corps) in a series of four letters dated 9/7/2000, 9/21/2000, 9/25/2000, and 9/27/2000. The purpose of this letter is to record our review comments on the December 2000 version of the documents listed below.

- "Comprehensive Stormwater Management Plan; Seattle-Tacoma International Airport Master Plan Update Improvements" dated December 2000 by Parametrix, Inc. Also reviewed were the separately-bound (as Volumes 2 through 4) Comprehensive Stormwater Management Plan Appendices A through Z dated December 2000. (SMP)

- "Natural Resource Mitigation Plan; Seattle-Tacoma International Airport; Master Plan Update Improvements" dated December 2000 by Parametrix, Inc. Also reviewed were the separately-bound Natural Resource Mitigation Plan Appendices A-E Design Drawings dated December 2000. (NRMP)

- "Wetland Functional Assessment and Impact Analysis; Master Plan Update Improvements; Seattle-Tacoma International Airport" dated December 2000 by Parametrix, Inc. (WFA)

Our qualifications to perform this review were described in our letter of November 24, 1999, and are repeated here. Mr. Rozeboom has over 20 years of specialized experience in surface water hydrology and hydraulics, including over 6 years as principal reviewer of all Master Drainage Plan, Stormwater Management Plan, and Storm Drainage Technical Information Report documents for the 1,300-acre Snoqualmie Ridge project currently under construction in the city of Snoqualmie. The Snoqualmie Ridge project is similar to the 3rd runway project in that it is a large site development which is subject to the requirements of the Washington State Department of Ecology Stormwater Management Manual and the King County Surface Water Design Manual (KCSWDM). Dr. Leytham has over 20 years of specialized experience in surface water hydrology and hydraulics, including serving as technical advisor to King County on flow control aspects of the 1990 and 1998 versions of the KCSWDM. Dr. Leytham was also responsible in 1990 for the original development of the Miller Creek basin HSPF simulation model which has since been modified by others for purposes of 3rd runway impact assessments and facility designs. Vitae for Mr. Rozeboom and Dr. Leytham are attached for reference.

Our review of the current Stormwater Management Plan and related documents has identified numerous technical deficiencies in the analyses and preliminary designs which present a risk of significant adverse impacts to the natural stream and wetland systems if the current documents are approved as a basis for mitigation of project impacts. The risk of adverse impacts is heightened by uncertainty over what performance standards will be eventually negotiated and applied for the final design of stormwater facilities, and the absence of a process for regulatory review of final drainage design plans for this large and complex project.

Our comments follow.

1. There is no clear and consistent definition of stormwater control standards to which the Port has committed to adhere. Although the SMP describes storm water control standards and target flow regimes at some length in Chapter 2 of the SMP, the standards discussed appear to still be under negotiation with Ecology. Ecology's current proposal to modify the NPDES permit[1] for SeaTac International Airport would extend permit coverage to stormwater discharges associated with the Third Runway and Master Plan Update projects. However, in the draft of the modified permit, project stormwater detention requirements are specified in Special Condition S14 as, "*All construction actions taken by the Permittee shall provide sufficient detention and/or shall use existing available detention capacity, in accordance with the Stormwater Management Manual for the Puget Sound Basin or its approved equivalent, to prevent an increase in the peak flow rate or flooding frequency of Miller Creek and Des Moines Creek.*" The problem with this language in the draft permit is that it specifies (requires?) a stormwater standard for the Third Runway and Master Plan Update

[1] Ecology held a February 12, 2001 public hearing on the proposed modification to NPDES Permit No. WA-002465-1. The deadline for written comments on the proposed modification is February 26, 2001, which is 10 days after the deadline for public comments on the Section 404 Permit application for the same project.

projects which is less stringent than the SMP "updated" detention standards (SMP section 2.1.4) sought by others at Ecology as a condition of Section 401 Certification for those same projects. The December 2000 SMP (page 6-3) indicates that "*the hydraulic design of the facilities will be reevaluated and detention volumes adjusted as appropriate to ensure that the Port's stormwater management standards are met.*" However, the "Port's stormwater standards" appear to be defined by the SMP (page 2-1) as being "in the King County and Ecology Manuals" and those manuals do not describe or require the "updated" detention requirements found in SMP Section 2.1.4. These inconsistencies in proposed standards are of concern and lead us to question whether the Port will implement designs per the updated standards cited in the current SMP or is anticipating future negotiations which will allow the facilities to be reevaluated and detention volumes to be reduced per the less stringent standards in the King County Manual or as required by the NPDES permit.

2. The lack of detailed stormwater plans, plus the lack of a clearly-defined review process for this very complex project, makes it likely that post-SMP detailed engineering and revisions to stormwater facility designs will fail to meet Ecology and King County performance expectations. The recent history for this project, particularly the major flaws in both the November 1999 and August 2000 versions of the project SMP, highlights the need for an independent design review to supplement the Port's quality assurance and review processes. Lack of an established review process is a very major concern given that the current SMP does not establish exactly what facilities and hydraulic controls will be constructed.

   Stormwater drainage regulations for the project site are defined by the King County Surface Water Design Manual (KCSWDM) as adopted by the city of SeaTac. The KCSWDM begins (Chapter 1) by describing the drainage review procedures necessary to implement the King County surface water policies and to ensure compliance with the manual's technical requirements. However, the Port has consistently claimed to be exempt from the KCSWDM drainage review requirements as well as all other KCSWDM "procedural" requirements[2]. The proposed project will have a long timeline and there will likely be a need for design adjustments to address unanticipated conditions which arise in the future. Without explicit descriptions in the SMP of the facilities and hydraulic controls to be constructed, plus certainty of ongoing, independent, competent review, there can be no reasonable assurance of project compliance with either King County surface water policies or Ecology conditions of approval for Section 401 Certification.

3. KCSWDM Core Requirement 7: Financial Guarantees and Liability. (Similar to Ecology's Minimum Requirement #11.) The objective of this "procedural" core requirement is to ensure that development projects have adequate financial resources to fully implement the stormwater management plan and that liability is not unduly incurred by local governments. The present SMP does not address the costs of the proposed improvements or offer any financial guarantees. Using costs presented in SMP Appendix M, a single 12.6 acre-foot

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[2]Procedural issues were previously raised in our comment letter dated November 24, 1999. The Port's response to those comments, in a "Response to 401/404 comments" document dated March 10, 2000, stated that the Port's Interlocal Agreement with the City of SeaTac includes an exemption from "specific County permitting procedures." In the same document, the Port response to our comment on drainage review requirements begins with the statement, "This comment refers to a procedural process that the Port is not obligated to follow."

vault for water quality treatment would cost $7,258,675 or about $13 per cubic foot. SMP page 6-5 shows that a total of 207.2 acre-feet of stormwater vaults are proposed. At $13/cubic foot, the proposed stormwater vaults alone would cost over $117,000,000. The SMP does not address or satisfy the applicable King County and Ecology requirements for financial guarantees, and provides no assurance of sufficient funding to construct the facilities being proposed.

The importance of costs and financing is also cited in a letter report dated November 10, 1999 to the US Army Corps of Engineers by Keith Macdonald, Ph.D., of CH2M Hill, who was hired by the Port to "prepare an objective, independent, peer review of the natural resources mitigation program" for the proposed Master Plan Update Improvements. Dr. Macdonald states that "Obviously, the success of the mitigation depends on the effectiveness of implementation and monitoring. . .It is critical that sufficient guaranteed funding be available. . ."

4. Sizing of stormwater facilities has relied on unsupported assumptions regarding future Industrial Wastewater System (IWS) capacity for processing airport runoff without overflows to the natural creek systems. If these assumptions are not achieved, the stormwater facilities proposed in the SMP may be undersized. The core questions are whether the IWS storage lagoons can be significantly expanded as has been proposed[3], and what future processing rate can be achieved. SMP page 7-15 indicates a requirement for AKART (all known available and reasonable methods of treatment) recommendations for handling of IWS flows to be fully implemented by June 2004, and that the recommended alternative is for IWS treated effluent to be discharged to a King County DNR facility at Renton. An important implication of this AKART requirement is that the current IWS configuration and capacity discussed in the SMP (Section 4.2.2.2) may be largely irrelevant to the future IWS configuration and capacity. According to the SMP, negotiations are ongoing for determining (future) IWS pre-treatment standards, flow limits and timing and other issues. The Storm Drain System (SDS) is being sized to accommodate year 2006 conditions and therefore needs to be compatible with the year 2006 IWS system which meets AKART requirements.

Proposed lagoon expansion is incompatible with safe airport operations. The FAA has published guidelines in Advisory Circular 150/5200-33 dated 5/1/97, titled "Hazardous Wildlife Attractants on or Near Airports." The proposed expansion of IWS Lagoon 3 falls under the Advisory Circular's definition of a wastewater treatment facility (definitions are given by SMP page 4-7). Section 2 of the Advisory Circular, "Land Uses that are Incompatible with Safe Airport Operations" recommends that any new wastewater treatment facilities or associated settling ponds be sited no closer than 10,000 feet from turbine aircraft movement areas. The existing third lagoon is located within 2,000 feet of the runway, and the proposed new expansion area is within 3,000 feet of the runway. The proposed expansion of the lagoon facilities, as assumed for purposes of SMP facility design, appears

[3] SMP Table 4-5 shows that the proposed expansion of IWS Lagoon 3 will add about 145 acre-feet of total storage. This significant volume is equal to about 45% of all other new stormwater storage volume proposed per SMP Table 6-2.

to be in direct conflict with these FAA guidelines which have been applied elsewhere in the project to preclude on-site mitigation for loss of wetlands.

Feasibility of proposed IWS discharge rate is not established. The future processing rate to be achieved by the IWS system is a variable which has yet to be designed and/or negotiated. Based on system performance predictions in the latest IWS design report[4], it is clear that consideration is being given to a processing rate which is substantially less than the 2.4 to 4 MGD treatment rates examined in the SMP (Table 4-2).

The IWS storage volumes which are assumed in the SMP presume that Lagoon 3 will be expanded from its current volume of 26 MG to a future volume of 72 MG. That future volume is not proposed or described in the IWS design report. Instead, the design report (page D-1) indicates that the required lagoon size is dependent on the available release rate--a 47 MG lagoon would be required for a release rate of 4 MGD while a larger 67 MG lagoon would be required for a release rate of 2 MGD. The report does not indicate what release rate would be associated with a 72 MG lagoon. The proposed expansion to 72 MG is understood to have been established as simply "the maximum possible capacity within the available area[5]."

The IWS design report provides information to suggest that there are benefits to having a lower processing rate. The report (page 4-4, Alternative A3) cites a major cost incentive for having a reduced IWS processing rate of 1 MGD in that effluent "can be metered to KCDNR at a controlled rate during off-peak hours, which is an operating benefit to KCDNR and a cost savings to the Port. . . the annual operating costs are approximately half of Alternative A1[6]: $2.9 million versus $5.8 million." The IWS design report however does not identify what size of lagoon would be required, for a 1 MGD processing rate, to prevent overflows into the SDS or directly into Des Moines Creek.

Due to an apparent conflict with FAA guidelines, it is uncertain whether the IWS lagoon capacity can be significantly expanded as has been assumed. Because of the unknown outcome of future negotiations between the Port and King County DNR, it is uncertain what future IWS release rates will be permitted, and whether any emergency/flood-event restrictions might be imposed on IWS releases[7]. These uncertainties are problematic for

[4] "Addendum to IWS Engineering Report" dated April 1998 by Kennedy/Jenks Consultants.

[5] Information provided by email from Ecology (Chung Yee), with reference to a letter dated November 10, 1999, from Michael D. Feldman of the Port to Kevin Fitzpatrick of Ecology.

[6] Alternative A1 involves enlarging Lagoon 3 to 47 MG and discharging 4 MGD to King County. Disadvantages to Alternative A1 include :" Very high annual operating costs for the first 20 years. . ." and " A new pretreatment permit with KCDNR must be obtained and complied with."

[7] Other documents obtained for review purposes (not part of the SMP) included sizing calculations for Lagoon #3 dated February 2000 by Kennedy/Jenks Consultants. That document discussed several "additional considerations" to support construction of a lagoon with more storage volume, including: "Downstream system owners may prohibit IWS flows from being released during high-flow events."

ensuring the adequacy of the proposed stormwater system because IWS capacity has a direct impact on the size of required stormwater facilities, yet the IWS system is being designed and permitted through processes which appear to be largely independent of the design and review processes for stormwater system planning. In the presence of these uncertainties, there can be no reasonable assurance that water quality standards will be met.

5. Problems similar to those resulting from SDS-IWS interdependence above are also found in a need for coordination between SDS facilities and low flow augmentation facilities. Specifically, a new proposal for reserve storage to augment low streamflows appears to have been added at the SMP at the last minute. SMP page 6-6 references "managed release of stormwater from reserved storage" but the summaries of stormwater facility volumes (SMP Table 6-2 and equivalent tables in other documents) do not contain any allowance for "reserved storage." The SMP is internally inconsistent in that the SMP page 6-6 list of factors which would mitigate low flow impacts fails to include the proposal from SMP page 6-10 that water for low flow augmentation will come from a well within the Tyee Valley Golf Course. Significant problems with SMP underestimation of low flow impacts and overestimation of mitigating factors are identified in other comments later in this letter. This comment focuses mostly on the unaddressed practical challenges of adding reserve storage capabilities to already-large stormwater facilities.

Under the current proposal for streamflow augmentation (from the Low Streamflow Analysis, pg 15), the Port will construct "additional storage volume in the base of selected detention facilities" to store winter season runoff until needed to support low flows during the dry season. The Low Streamflow Analysis (pg 20) further indicates that about 16.0 acre-feet of reserve storage would be required to mitigate for estimated low flow impacts. (In other comments we describe why low flow impacts have been underestimated.) Several of the proposed detention facility exhibits presented in SMP Appendix D do have some "dead storage" capacity for reserve storm water release, but the total storage (based on spot checks) appears to fall short of the target amounts. There is no tabulation on the exhibits or elsewhere of how much stormwater reserve is to be provided in total or at each facility: our spot checks required estimation of volumes from facility dimensions. A check of Vault G1 (Exhibit C151) found that the design detention volume (9.2 acre-feet) would not be available given the facility dimensions and the depth of water being allocated to dead storage. Operation of these facilities may be impractical as now configured. For example, a valve box to control reserve releases from Vault G1 would need to be either buried at about 35 feet depth (hard to operate) for runway-grade access or, for a more reasonable shallow depth, the valve box would need to be accessed and operated from a difficult-access ledge on the embankment terrace. The deepening of the vaults to provide reserve storage has caused some vaults to exceed King County maximum cover requirements and will necessitate special designs to ensure structural integrity. The reserve (dead) storage layer at the base of the detention facilities function will accumulate and concentrate settleable solids and particulate-based pollutants from the airport stormwater runoff; that "dead storage" water would later be released under very low-flow conditions with little or no opportunity for dilution of any concentrated pollutants. There is also a potential for development of anaerobic conditions in the dead storage zone which would further worsen the quality of the "reserved" water. Our point is that the "reserve stormwater" plans are new to the SMP design/review process. They are at a highly preliminary stage of development and require significant further work prior to a detailed design review which could offer any assurance that the plans are feasible or capable of providing useful low-flow mitigation.

6. While it appears that many of the gross inconsistencies in previous HSPF models have been resolved, we remain surprised by the lack of checks on the hydrologic simulation results and lack of effort to explore apparent data irregularities. This comment focuses on calibration deficiencies for Des Moines Creek.

The hydrologic model calibration report for Des Moines Creek indicates (SMP pages B1-13 and B1-14) that model results under-simulate recorded base flows at both of the upper-basin gages used for model calibration. The justification offered for under-simulation of inflows at Tyee Pond is a speculative "*it seems unlikely that enough rainfall can get into groundwater to support 0.35 base flow*" and a presumption that the stream should be gaining water in its lower reaches. The explanation offered for under-simulation of flows at the SDS3 outfall is that "*it is unknown what phenomenon could produce this base flow. One explanation is that the flow monitoring device will not register zero flow.*" In our opinion, further efforts should be made to evaluate the reliability of the available data. In the case of the SDS3 gage, we are unaware of any flow monitoring devices which, properly installed and maintained, would fail to register zero flow. Failure to register zero flow, if true, could reflect a problem with the gage and should be explored to determine if there are also problems with the high-flow data being reported from the gage. Given the questions over low flow calibration for both the East Branch (Tyee Pond) and West Branch (SDS3) tributaries to Des Moines Creek, the model results should be checked against the low flow data which are available for King County Gage 11F, Tyee Weir, below the confluence of these headwater streams. The calibration report does include one plot of peak daily flows at a "Golf Weir" but we could not locate any discussion of those results.

There are inconsistencies and problems with the Des Moines Creek model treatment of area groundwater conditions represented by Figure B1-3. The calibration report text (pg B1-10) indicates inflow of groundwater from 1,240 acres of area which is noncontiguous with the surface watershed; this is inconsistent with the model input sequence which has only 512 acres. Also, our independent measurement of the Des Moines Creek noncontiguous area (per Figure B1-3) yielded about 850 acres of total area. Another groundwater-related problem with calibration is that it has overlooked possible stream losses to groundwater in the lower part of the basin. Figure B1-3 groundwater mapping shows that the Des Moines Creek below about elevation 200 feet does not intersect the regional groundwater table. This transition area corresponds roughly to the location of a knickpoint described in SMP page P-2 where the Des Moines Creek channel gradient increases and where bed sediments change from fine grained materials to relatively coarse materials with boulders, cobbles, gravel, and fine sand. Considering the evidence of the streamflow data, it seems likely that the lower part of Des Moines Creek includes a "losing reach" which has cut through the perching layer which supports the regional shallow groundwater table. The physical condition of a losing reach would be consistent with streamflow data at the mouth which show unexpectedly low flow peaks and volumes relative to streamflow data for the headwater areas. It is possible that the "poor calibration" problems described by SMP page B1-13, and the difficulty in reconciling measured flows at the upper and lower gages, could be rectified if the presence of a losing reach were confirmed.

We recognize that model calibration is a challenging process and that data reliability is often an issue. However, because the purpose of this work is to address and mitigate conditions in the upper basin (airport) areas of the watershed, calibration efforts should place more emphasis on matching upper basin flows unless those data are confirmed to be unreliable. The current calibration effort is deficient because it has placed too much emphasis on matching conditions at the lower gage, and has prematurely discounted the more-important upper basin data.

7. In our letter of Sept 21, 2000, we pointed out that the modeling had not made any use of King County stream gage 42C which measures flows in Tributary 0371A (a.k.a. Walker Creek) near 281 S 171st Place, a short distance downstream from the Walker Creek wetland. That gage provides direct information on flows in the headwater reach of this stream below the area of the proposed 3rd runway, and is more meaningful than the lower gage near the mouth for calibrating a streamflow model which is intended to examine streamflow effects of the 3rd runway. However, in the December 2000 SMP, there is again no mention or use of the available stream gage data for upper Walker Creek. The calibration is deficient for its failure to use this readily available streamflow data.

8. The Walker Creek calibration for low flows was achieved with a model adjustment which appears to be inconsistent with actual basin characteristics. In order to simulate flow volumes (and low flows), the Walker Creek model (SMP page B2-51) has included groundwater flows from 630 acres of till grass lands located in the (surface topography) Des Moines Creek basin, based on groundwater mapping shown by SMP Figure B2-23. However, our review of the same groundwater mapping does not show support for this acreage. We have measured the identified "Noncontiguous Walker Creek groundwater area" to be only about 690 acres in total, before adjustment for impervious surfaces. From Figure 2-1 and aerial photos, probably about one half of that total area consists of impervious surfaces which should be collected in either the IWS or other piped storm drain system and should not be available for groundwater recharge. These data checks indicate that the groundwater recharge area required (630 acres) to balance the measured Walker Creek flows is much greater than the available groundwater recharge area (about 350 acres) indicated by the available mapping. We do not know if the difficulty in simulating sufficient flow volume in Walker Creek is related to apparently similar problems in reproducing recorded flow volumes in the upper Des Moines Creek basin.

It is possible that base flows in the model calibration period have been supported in part by leakage from the IWS conveyance system and by seepage from unlined IWS lagoons. It is also possible, although more speculative, that irrigation runoff from the golf course may be influencing the base flows. It is difficult to provide any reasonable assurance of appropriate mitigation for airport impacts on stream base flows, or seepage flows to wetlands, when the source of those flows is so poorly understood.

9. The SMP model calibration of airport fill parameters appears to be biased towards parameters which understate the hydrologic flashiness of the fill which is being placed. Airport fill calibration is described in SMP (Appendix) page A-16; calibration results are plotted on page 4 of Attachment B to that appendix. The calibration data show that the model does a good job of representing average flows, but does not cover the full range of flows which were measured during the calibration period.[8]. Peak flows are consistently (in 5 out of 6 events) underestimated, and low flows are consistently overestimated (by about 0.03 cfs from the 20-acre fill site being assessed). One consequence of these calibration results is that stormwater detention facilities might be slightly undersized. A second consequence of these calibration results is that any assessment of runway fill impacts on base flows, using HSPF modeling with these calibration parameters, might underestimate actual base flow impacts.

10. The SMP and related documents fail to consider the impacts to low flows in Des Moines Creek and Walker Creek which will result from recent lagoon lining improvements to the IWS system. The IWS has a direct significant impact on seepage and base flows in the Walker and Des Moines Creek systems by its removal of large areas of basin which would naturally form the headwater recharge areas for those streams. Until recently, the effects of these diversions have been partially offset by infiltration recharge to groundwater from the three IWS storage lagoons which are located near the groundwater divide between Walker and Des Moines Creeks.

Our source of information on the history and status of the IWS system is a recent hydrogeologic study by Associated Earth Sciences, Inc., "Hydrogeologic Study, Industrial Waste System (IWS) Plant and Lagoons, Seattle Tacoma International Airport," prepared for Port of Seattle, June 21, 2000. Lagoon 1 has been used to store wastewater since 1965. Lagoon 2 was built in 1972 and "is utilized during times of heavy rainfall events." Lagoon 3 was constructed in 1979 and "is used to provide excess storage capacity for industrial wastewater in the event that Lagoons 1 and 2 reach capacity." The bottoms of the lagoons most regularly in service - Lagoons 1 and 2 - were reportedly "composed of compacted gravelly sand" which should have a relatively high infiltration capacity. A program to install leak prevention liner systems in the lagoons has been underway since 1996: Lagoon 1 was lined in 1996, Lagoon 2 was lined in 1997, and construction documents have been prepared for Lagoon 3 to be lined in the near future. The flow augmentation recommendations in the 1997 Des Moines Creek Basin Plan were likely based on data which did not reflect impacts of the lagoon linings. Our point is that airport impacts to stream base flows, as well as mitigation needs, have likely been underestimated because they have not considered the effect of lining these lagoons.

[8] Calibration period was for 25 days in February 1999. According to NOAA-published rainfall data, SeaTac airport recorded approximately 5.6 inches during this period.

11. The SMP and related documents fail to consider the additional adverse impacts to streamflows in Des Moines Creek which will result from the proposed development of Borrow Areas 1, 3, and 4 as a source of 6.7 million cubic yards of fill for the 3rd runway. Information on the proposed borrow area development is found in the Appendices C and D of the Port's December 2000 Wetland Functional Assessment and Impact Analysis,[9] and in Ecology's June 2000 Sea-Tac Runway Hydrologic Studies Report by Pacific Groundwater Group (PGG). The three borrow area sites have a combined area of approximately 217 acres and are proposed to be mined to depths as great as 100 feet below existing grade. The material to be excavated is described as glacially-deposited, slightly silty to silty sands and gravels (outwash soils).

Airphotos of the airport vicinity show that the existing land use at the borrow areas is primarily forest. Land use for these areas (a.k.a. South Borrow Area, Onsite Borrow Source Areas 1-4) is further described in the project 1996 FEIS Appendix M, pages M-2 and M-3 as "Both upland and wetland second-growth deciduous forest are prevalent components of the South Borrow Area" and "Upland coniferous forest is found in the northwest corner of the South Borrow Area."

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[9] Appendix C is a Hart Crowser memorandum dated December 8, 2000 regarding "Third Runway Project; Borrow Areas 1, 3, and 4; Projected Impacts to Wetlands." Appendix D is a Hart Crowser memorandum dated October 20, 2000 regarding "Sea-Tac Third Runway - Borrow Area 3 Preservation of Wetlands."

Development (excavation) of the borrow areas will eliminate most of the remaining forest[10] in the headwater areas of Des Moines Creek. There will be several impacts to streamflows in Des Moines Creek as a result of physical impacts of the excavation work. First, the cutting of the forest and stripping the land of forest duff and organic soils will produced increased runoff volumes as well as increased peak flows. Second, depending on the eventual site grading and soils, infiltration and groundwater recharge may be reduced relative to the current forested condition. Third, summer base flows in Des Moines Creek can be expected to be impaired due to lost flow attenuation capacity, just as summer base flows impacts in Miller Creek are expected to be moderated somewhat by flow attenuation effects in the embankment fill. Finally, base flow contributions to Des Moines Creek from the borrow areas could be significantly affected if the excavations should strip away outwash materials to leave a surface exposure of till soils or if excavations should penetrate any groundwater perching horizons.

PGG Figure 4-2 shows a cross section for Borrow Area 1. Surface geology consists of a 5- to 25-foot depth of (permeable) recessional soils overlying a (relatively impermeable) till layer which is typically about 30 feet thick. Under current conditions, very little surface runoff would be expected. Precipitation in excess of the amount consumed by forest evaporation and transpiration would infiltrate through the recessional soils, encounter the till perching layer, and gradually seep laterally to provide seepage/base flow to Des Moines Creek. Grading and excavation will cause both the forest and the recessional soils to be removed from this area. The remaining (newly-exposed) surface geology will instead consist of till which will generate relatively large surface discharges (high peak flows) and relatively little seepage or base flow. Long term impacts will also be influenced by undetermined site restoration activities or conversion to non-forest land use.

PGG Figure 4-3 shows a cross section for Borrow Areas 3&4. Surface geology is variable. In the area of Borrow Area 3, which is closest to Des Moines Creek, the surface geology consists of a typically 10-foot depth of (permeable) recessional soils overlying a quite thin (less than 10 feet) lens of relatively impermeable perching layer. The current hydrologic response for the area of Borrow Area 3 would be similar to that described above for Borrow Area 1. In the area of Borrow Area 4, the surface geology consists of a thick (up to 100 feet) depth of advance outwash soils overlying a perching horizon. The perching horizon beneath Borrow Area 4 connects with the perching layer beneath Borrow Area 3, such that the seepage flows from both areas eventually merge and flow (seep) together en route to Des Moines Creek. The current hydrologic response for the area of Borrow Area 4 would be generally similar to that for Borrow Areas 1 and 3 except that there would be even greater flow attenuation due to the thickness of the outwash deposit and the greater distance from Borrow Area 4 to Des Moines Creek.

The proposed excavation of Borrow Areas 3 and 4, as proposed, may leave a surface exposure of deep advance outwash soils. This soil exposure (assuming no conversion to land use with impervious surfaces) should not cause any increase in surface flows and the elimination of the forest cover will promote increased groundwater recharge. However, the

[10]Additional forested basin will be lost by development of the SASA element of the Master Plan Update Improvements.

proposed grading will penetrate and remove a perching layer which may currently be conveying borrow area seepage flow to the headwaters of Des Moines Creek. As a result, the base flow from these borrow areas to the upper reaches of Des Moines Creek may be significantly diminished.

In summary, the proposed development of the borrow areas is likely to result in adverse permanent impacts to Des Moines Creek, including increased peak flows and reduced base flows, which have not been assessed and for which no mitigation has been proposed.

12. There are numerous shortcomings in the evaluation of the potential low stream flow impacts described by SMP pages 6-5 and 6-6. Our comments below reference the source of that analyses which is the December 2000 Earth Tech report, "Seattle-Tacoma Airport Master Plan Update Low Streamflow Analysis."

a) The low flow analysis does not provide information to indicate the accuracy of the HSPF model in simulating low flows. Data provided in Table 1 for recorded average flows in August and September are for relatively-short periods of available record. Data provided for simulated average flows in August and September are for a much longer (1949-1996) period of simulation. These data sets are not directly comparable due to different periods of record. The report needs to provide a summary of simulated and observed monthly flows for periods of recorded data.

b) The report does not include HSPF input sequences to confirm what land uses and basin boundaries were assumed for any of the Des Moines or Walker Creek analyses. For Miller Creek, HSPF input sequences were provided only for year 2006 post-development conditions. In light of the major modeling discrepancies found in the previous SMP, and the fact that the present work is being conducted by three separate firms, it is important to confirm what models were used for each of the analyses.

c) As indicated in our above comments, model calibration appears to have relied on faulty measurements of groundwater tributary areas which are noncontiguous with the surface water basins (Figures B1-3 and B2-2). Walker Creek calibration relied on groundwater inputs from about 630 acres of noncontiguous pervious basin; however only about 350 acres of noncontiguous pervious basin appears to be actually available. There is also an apparent inconsistency in the modeling of noncontiguous groundwater inputs to Des Moines Creek: the text (SMP pg B1-10) indicates 1,240 acres but the model input file uses 512 acres. These inconsistencies need to be resolved if there is to be any confidence in model predictions regarding project effects on low flows.

d) Project impacts to low flows in areas of runway fill (Miller and Walker Creeks) may be underestimated because the HSPF model parameters used to simulate the fill materials produce larger low flows than indicated by the available calibration data. (See calibration plot, SMP Appendix A, Attachment B, Page 4. Wet season low flows are consistently overestimated by about 0.03 cfs from the 20-acre fill site being assessed.)

e) Project impacts to low flows in Des Moines Creek and Walker Creek have been underestimated because the assessment has ignored the post-1994 effects of lining the IWS storage lagoons.

f) Project impacts to low flows in Des Moines Creek have been underestimated because the assessment has ignored the post-1994 expansion of the IWS system by about 111 acres (per SMP page 5-4) and corresponding reduction in the Des Moines Creek tributary basin. The IWS basin expansion (Des Moines Creek basin reduction) is not reflected by the available supporting data for the low flow study Instead, the area summaries presented with the Low Flow Study, Appendix D, Figure 3 indicate that the tributary basin to Des Moines Creek will increase by about 16 acres from 1994 to 2006.

g) Project impacts to low flows in Des Moines Creek have been underestimated because the assessment has ignored the effects of the loss of forest and excavation of 6.7 million cubic yards of outwash material from proposed borrow area sites at what are now **the forested headwater areas of the basin.**

In summary, insufficient information has been provided to confirm what models were used for the low flow analysis, or to establish whether the models are reasonably well calibrated for assessing low flows conditions. Furthermore, the analysis methods have overlooked several airport activities which will likely have an adverse impact on low streamflows, particularly in the Des Moines Creek basin. Individually and cumulatively, these problems result in a failure to adequately address airport impacts on low streamflows and associated water quality concerns in the affected streams, and a corresponding failure to provide reasonable assurance of adequate mitigation.

13. Estimates in the Low Streamflow Analysis (pages 5 through 9) of the mitigating effects of "Fill Infiltration Discharge" are inconsistent with the measured hydrologic response of the 1998 fill embankment as shown in SMP Appendix A. The measured runoff from the embankment indicates a relatively rapid flashy response to rainfall with rapid recession rates which are inconsistent with the statement (Low Streamflow Analysis page 6) that fill " would provide increased discharge from the fill area during the critical low flow periods in area wetlands and streams". One of the principal problems appears to be that the PGG study[11] used as the basis for this assessment assumed a theoretical hydraulic conductivity for the fill which is far greater than the infiltration capacity which can be inferred from either the measured data or the HSPF model calibration. The PGG study (page C-4) assumed a hydraulic conductivity for the fill of 1.35×10^{-4} cm/sec (equivalent to 0.19 inches/hour) based on theoretical values for fill gradation specifications. That theoretical value is significantly greater than short term rainfall intensities associated with production of runoff during the monitoring period, and is nearly 10 times greater than the nominal infiltration rate of 0.02

[11] Pacific Groundwater Group, "Sea-Tac Runway Fill Hydrologic Studies Report," for Washington State Department of Ecology, June 19, 2000.

of water volume (detention storage plus reserve storage) with a water depth of 30 feet. There is an obvious need for a safety review to assure the structural stability of Vault SDS7. Our concerns over Vault G1 result from its close (about 20 feet) proximity to the top edge of a 140-foot high fill embankment. Furthermore, because of its proposed placement in fill, Vault G1 (and perhaps others) fails to satisfy the KCSWDM technical requirement (pg 5-37) that "Vaults shall not be allowed in fill slopes, unless analyzed in a geotechnical report for stability and constructability."

18. Many of the proposed vaults are in violation of KCSWDM pg 5-38 which specifies, "The maximum depth from finished grade to the vault invert shall be 20 feet." This requirement appears to relate to the maximum loading which a conventional vault structure can withstand without risk of structural failure. If so, then special structural designs will need to be developed for Vaults SDS3 and G1 (cover depth to about 40 feet), Vaults SDN3 and C1 (cover depth to about 30 feet), and Vaults M6 and C2 (cover depth to about 25 feet). Due to the currently-proposed depths, none of these six vault facilities are in compliance with the King County technical requirements for stormwater facilities. In some cases, this compliance problem has been caused or worsened because the facilities have been enlarged (deepened) to accommodate reserve stormwater storage for purposes of low flow augmentation. Further analysis is necessary to determine whether these facilities are viable.

19. SMP section 3.1.2.3 discusses concerns with standing open water. A drain time calculation proposed in the SMP for addressing open water concerns is inappropriate and will underestimate actual open water durations. The drain time method is inconsistent with actual prolonged-duration precipitation conditions in the Puget Sound. Continuous simulation methods need to be used. (Also see Comments 10 and 11 of our letter of November 24, 1999.) The current SMP proposes an inappropriate methodology to assess open water durations and furthermore fails to provide any analysis, by any method, of expected open water durations in any of the stormwater facilities being proposed. The consequence of using an inappropriate analysis methodology in this instance is that the duration of standing open water is likely to be significantly underestimated and that mitigation designs (for example netting over lower cells within detention ponds) could fail to prevent the creation of open water waterfowl attractants which are incompatible with safe airport operations.

20. Insufficient information has been provided regarding proposed Erosion and Sediment Control (ESC) facilities to offer any assurance that facilities are adequately sized and will perform as intended. There is no cogent explanation of how this ESC system is supposed to function and there are numerous potential problems inherent in the current SMP plans.. Our concerns are heightened because the Port has already issued "Third Runway- Embankment Construction Phase 4" construction plans[13] and specifications for erosion control facilities and some permanent drainage facilities, without any known independent review or approval

[13]Port of Seattle major contract construction plans titled "Third Runway - Embankment Construction - Phase 4", Work Order #101346, Project STIA-0104-T-01, were approved on 1/25/01 by Raymond P. Rawe, Director of Engineering Services. The accompanying two-volume Project Manual, including Specifications, prepared under the direction of Raymond P. Rawe, is dated January 29, 2001.

of those plans by any regulatory agency. Further review, prior to project approval, is needed to resolve the following questions:

a) Where are the clearing limits for the proposed work? King County core requirement 1.2.5.1 requires that prior to any site clearing or grading, areas to remain undisturbed during project construction shall be delineated. For example, SMP Appendix R, Exhibit C24 suggests that there will be an undisturbed strip, which includes some wetlands, between a line marked "limits of embankment" and a proposed TESC ditch some distance downhill. Is this strip supposed to remain undisturbed? On the corresponding grading and drainage plan for the same area (SMP Appendix O, Exhibit C115) there are again no work limits shown and the plans are deficient for not identifying the grading necessary to restore the wetlands which were altered by construction of TESC facilities.

b) What is the tributary area for each of the proposed ESC facilities? What are the design flows? Have the design calculations been reviewed? Who was responsible for this review?

c) How big are the pumps being proposed for this work? (Pumps need to be of sufficient capacity and compatible with ESC processing rates and storage volume.) What is the power supply for these pumps? If gas/diesel pumps (or power generators) are proposed, how will refueling be accomplished and what safeguards will be in place to contain spills?

d) How long will these "temporary" facilities be in place. One year? Six years?

e) How are the "outer swale" ditches supposed to work? According to the geotechnical engineering report (SMP Appendix L, Figure 8) these ditches are supposed to intercept the seepage flow from the base of the embankment and convey the water to wetlands. Collection of the (clean water) seepage flow is in conflict with the use of these same ditches for conveyance of (turbid water) construction site runoff as proposed in the SMP Appendix R exhibits. Capture and routing of clean water seepage flows to erosion control facilities might overload sediment pond processing capacity, causing releases of untreated turbid water during storm events. Capture and routing of clean water seepage in interceptor swales would furthermore cause downslope wetlands to be significantly de-watered during the (multi-year?) period of construction.

f) Why is temporary Pond A being excavated to a depth of approximately 10 feet in the middle of a wetland? The pond location is shown by SMP Appendix R Exhibit C24; greater detail is shown on Phase 4 construction drawings. The construction drawings include a note warning the contractor to anticipate seasonal groundwater at about 1 to 1.5 feet below ground surface. It is unrealistic to expect that a simple geotextile membrane as proposed will succeed in keeping the surrounding groundwater out of this pond. It is probable that the pond will be constantly recharged by the wetland water supply and that pumping from this pond will be functionally equivalent to pumping from the wetland. In addition to adverse impacts on the wetland, it is likely that ESC facilities have not been sized to accommodate this water.

The above questions result in part from a failure to recognize or satisfy the procedural, design review provisions of the King County and Ecology requirements. In this instance, the lesser requirement is defined by Ecology's Stormwater Program Guidance Manual, which specifies that a development site of this size must prepare a Large Parcel Erosion and Sediment Control Plan[14], comprising both a narrative report plus site plans, to demonstrate compliance with minimum requirements. The current erosion control site plans do not demonstrate compliance with minimum erosion control requirements, and give rise to numerous concerns which, individually and cumulatively, create a significant risk of recurring uncontrolled releases of construction site runoff.

21. The plans do not show how runoff from the face of the MSE wall, or from the face of the embankment, will be conveyed to the stormwater detention facilities. There are two issues. First, drainage must be provided from terraces on the face of the wall and the face of the embankment drainage in order to prevent erosion damage and to minimize the possibility of surface saturation which might result in localized slope failures. Second, this water must be conveyed to the stormwater detention facilities which will provide the required Level 2 flow control. Plans in SMP Appendix O, Exhibit C115 show that undetained surface runoff collecting at the bottom of the embankment, and also from the airport security road, would be discharged directly into adjacent wetlands without any peak flow detention as required by King County and Ecology regulations.

22. SMP Page 3-7 states, "*Several examples of water-induced slope failures have occurred recently, including one airport embankment project in Telluride, Colorado, that resulted in airport closure for one year. The slope failure was primarily attributed to stormwater build-up within the embankment.*" Because of the height of the proposed 3rd runway embankment, and the potentially catastrophic consequences of a slope or wall failure, the design documentation for the SeaTac project should identify the specific design and environmental factors which were associated with those failures. For example, were previous failures associated with poorly-draining fill materials, inadequate construction methods, or insufficient drainage systems? Were previous failures associated with specific climatic conditions such as unusually intense cloudburst events or an unusually prolonged rainfall event or closely-spaced series of intense events? Careful examination of the causes of known recent water-induced slope failures is a necessary, but missing, first step to ensure that the 3rd runway project does not repeat whatever errors or oversights may have been responsible for past slope failures.

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[14] See "Stormwater Erosion and Sediment Control for Large Parcel Construction", Department of Ecology Report WQ-R-93-012 1 #4 of 5. Also available at http://www.ecy.wa.gov/pubs/wqr93013.pdf

Based on our review of the Stormwater Management Plan documents, there are at least two drainage issues affecting the fill embankment which should be addressed and resolved prior to project approval.

a) There appears to be a significant discrepancy between the embankment theoretical infiltration properties assumed by geotechnical specialists responsible for the design of the embankment and the embankment infiltration properties inferred through stormwater runoff model calibration to data from the 1998 embankment by other specialists responsible for the design of stormwater management facilities. The geotechnical analysis of the embankment and wall, and design of internal drainage systems, should account for a range of worse-case scenarios which might result from variable (or uncertain) infiltration properties. For instance, if the unexpectedly-low observed infiltration capacity was suspected to be a result of periodic applications of tackifiers or emulsions or other surface treatments for erosion control during construction, then the embankment geotechnical analysis should anticipate perching horizons and saturated zones within the embankment. Review of past slope failures should consider whether discrepancies between theoretical and actual infiltration rates may have been a contributing factor.

b) Drainage from the steps in the wall and embankment should be designed to handle cloudburst rainfall quantities computed against the surface area of these features, rather than the plan view. It is not apparent that the SMP has given any consideration to either the specific scenario of wind-driven (non-vertical) precipitation or the more general surface runoff drainage needs for the face of the wall and embankment. Review of past slope failures should assess the role and significance of surface drainage from the face of the embankment (or wall) as a contributing factor.

23. The proposed construction excavation for Pond D, as shown by SMP Appendix D, Exhibits C133 through C134.1, is very likely to intercept the local shallow regional groundwater table and to significantly disrupt the water supply to Wetland 39. We question the accuracy of groundwater levels shown by Exhibit C134.1 which suggests the maximum seasonal water level in the vicinity of the pond would be slightly below the proposed pond bottom at elevation 336.0. There is strong evidence to suggest that the excavation proposed for Pond D, to depths as great as 25 feet below grade, will intercept the local groundwater table. First, the Hart Crowser study of local groundwater conditions (SMP Appendix L) found that the shallow groundwater table is typically 10 feet below existing ground level. Second, there is an existing surface expression of groundwater at Wetland 41a which is in the footprint of Pond D. Finally, it can be seen from Exhibit C133.1 that Wetland 39 (shown but not labeled on the exhibit) begins at about elevation 348 feet, 12 feet above the proposed bottom of pond.

24. The NRMP (page 3-10) asserts that compensatory storage will be provided to mitigate for approximately 5.24 acre-ft of floodplain storage which will be lost due to embankment fill. However, our review of the proposed design has found that the compensatory storage will fail to provide any mitigation for loss of storage during frequently-occurring flood events.

Loss of compensatory storage for frequently-occurring events (such as floods with return periods in the range of 2 to 10 years) might result in increased peak flows and erosion during those events.

Grading plans for the proposed compensatory floodplain area are shown by NRMP Appendix A, Sheet STIA-9805-C2. A hydraulic analysis for the associated reach of Miller Creek is presented in SMP Appendix J. The main problem with the proposed design is that the compensatory floodplain will be separated from the (relocated) stream channel by a ridge typically 2 to 4 feet higher than the floodplain. Also, the relocated channel will include a constructed 32-foot wide high flow section, independent of the floodplain, which will provide significant flow conveyance within the main channel. The ridge separating the main channel from the floodplain is apparent from the grading plans and also from NRMP Figure 5.1-6, titled "Typical Cross-Section of Miller Creek Floodplain Enhancement." The SMP hydraulic analysis shows that under major 100-year flood conditions this ridge (which has a top elevation of about 265 feet) is expected to be overtopped by depth of only about 0.5 feet. During less extreme events, the ridge will prevent floodwaters from entering the compensatory floodplain. There is no explanation for why a ridge is proposed which would prevent floodwater access to the floodplain mitigation area for all but extreme events. The compensatory floodplain design, as currently proposed, is insufficient to fully mitigate for the hydraulic effects of the embankment fill. The consequence, as stated above, is for increased peak flows and erosion during frequently-occurring flood events.

25. The proposed mitigation objectives for the Miller Creek relocation project are described by NRMP Table 5.1-2 (NRMP page 5-4). However, there are no calculations or other design information to demonstrate that the goals and design criteria will be accomplished with the design now proposed. From comparison of the December 2000 and August 1999 versions of the NRMP, we infer that some of the problems with the initial design have been recognized, but a revised design has yet to be developed which would accomplish the past or current performance objectives. The main problems are that the relocated channel is likely to go dry during low flow periods if it is constructed, as proposed, over a two-foot thick bed of highly-permeable spawning gravels. We notice that the design criteria in the December 2000 NRMP is to "Construct low flow channel 8 feet wide with 1:1 slopes and 0.5 ft deep to convey summer base flows" and does not identify a minimum flow depth which would prevent fish stranding. By contrast, the performance standard in the August 1999 NRMP (Table 5-1.1) was clearly established as a minimum flow depth of 0.25 ft at 0.5 cfs. We have commented previously that the proposed 8-feet wide channel will almost certainly not support a minimum flow depth of 0.25 cfs, especially if it is constructed over top of highly permeable gravels which will convey significant sub-surface flow. Another change between the August 1999 and December 2000 NRMP document is that the earlier (1999) design criteria was that "100 year flood flows will overtop the channel into the floodplain" whereas the current (2000) criteria is that "flows greater than the annual peak flow will overtop the channel and inundate the adjacent floodplain restoration." However, the hydraulic properties (width, slope, depth) for the relocated channel as shown in current design drawings (Appendix A to December 2000 NRMP) are essentially unchanged from the hydraulic properties as shown in previous versions of the design drawings. Our point is the NRMP

fails to provide any calculations to indicate that the proposed relocated reach of Miller Creek channel will accomplish its changing design objectives. Our independent review suggests that the channel design as now proposed will fail to accomplish performance goals for minimum depth of flow and for floodplain inundation.

In summary, there continue to be numerous deficiencies in the analyses and preliminary designs which present a risk of significant adverse impacts to the natural stream and wetland systems if the December 2000 versions of the Comprehensive Stormwater Management Plan and Natural Resource Mitigation Plan are approved as a basis for mitigation of project impacts. We request on behalf of the Airport Communities Coalition that, prior to regulatory certification or approval of the proposed 3rd runway project, the applicant be required to respond to the issues we have raised in this letter, and that we be granted the opportunity to provide follow-up review and comment on that response.

Sincerely,

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