APPENDIX 6

Groundwater Assessment

Douglas Partners
Geotechnics • Environment • Groundwater

Integrated Practical Solutions

REPORT
on
POTENTIAL GROUNDWATER IMPACTS
STAGE 4 PROJECT
PWCS KOORAGANG COAL TERMINAL

Prepared for

UMWELT (AUSTRALIA) PTY LTD

on behalf of

PORT WARATAH COAL SERVICES LIMITED

Project 49425

OCTOBER 2009

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23 October 2009

REPORT ON
POTENTIAL GROUNDWATER IMPACTS
STAGE 4 PROJECT, KOORAGANG COAL TERMINAL
KOORAGANG ISLAND

1. INTRODUCTION

This report presents the results of an assessment of the potential for impacts to groundwater associated with the Stage 4 Project comprising a fourth dump station and fourth ship loader at the Port Waratah Coal Services (PWCS) Kooragang Coal Terminal (KCT). The work was carried out for Umwelt on behalf of PWCS.

The proposed development includes excavation and dewatering associated with construction of a new dump station, construction of a new overhead conveyor to the wharf and a new ship loader. The purpose of the assessment was to provide the following information:

- Assessment of the existing groundwater environment in the project area based on previous studies;
- Consideration of the likelihood of actual or potential acid sulphate soils at the site and recommend appropriate management measures;
- Assessment of likely impacts of the project on groundwater quality through mobilisation / exposure of potential contaminants;
- Assessment of the likelihood of significant alteration of the groundwater regime due to the project;
- Recommended measures to manage and mitigate any identified groundwater impacts.

The scope of work included the following

- Review of available groundwater level data and groundwater contour plans to characterise the groundwater regime for identified activities;
- Review available groundwater chemistry data to characterise groundwater quality;
- Review of previous acid sulphate soil testing at the site and assess potential impacts;
- Review of existing subsurface conditions (soil and groundwater) at the proposed dump station site and comment on likely dewatering requirements and recommendations for management of dewatered water;
- Review of existing subsurface conditions (soil and groundwater) along the conveyor alignment and wharf area, and comment on likely impacts on groundwater (if any);
- Preparation of a report summarising the findings and recommendations.

2. PROPOSED DEVELOPMENT

2.1 General

It is understood that the Stage 4 project will comprise the following:

- Fourth dump station, associated rail facilities, sample plant and inbound conveyors. The new dump station and inbound conveyor will be located to the south of the existing dump stations;
- Augmentation to the rail loop to include an additional inbound and outbound track to the fourth rail receival facility;
- Transfer houses;
- Surge bins;
- Outbound sample plant;
- Ship loader conveyor;
- Fourth ship loader.

The components of the proposed development which would have potential to impact on groundwater would be those that involve elements that would potentially extend below the known water table. This includes the following:

- Proposed 4th Dump station and subsurface section of inbound conveyor to stockyard;
- Footings for proposed conveyor and transfer houses.

2.2 Dump Station Construction

The design of the proposed dump station will be consistent with the design of the existing dump stations, however the construction methodology has been altered to incorporate jet grouting to form the floor of the dump station, thereby substantially reducing groundwater dewatering requirements.

It is understood that the dump station will be approximately 15 m deep, 12 m wide and 66 m long. There will be two floors, the upper floor will have a base level of RL -3.8 m and the lower floor will have a base level of RL -10.7 m. A conveyor will exit the base of the dump station through a tunnel which will slope up to a surface exit point about 200 m to the east.

The main walls of the dump station will be constructed in-situ as diaphragm walls. Diaphragm walls are formed within an excavated trench supported by a bentonite slurry. Steel reinforcement is placed within the excavated trench and then concrete is placed in the base of the trench which displaces the bentonite slurry to the surface.

The base of the main dump station is then proposed to be formed by jet grouting. Jet grouting comprises installing a drill head to the target floor depth and then forcing cement grout into the sand at high pressure. A bulb of impermeable sand-cement is then formed around the drill head. An low permeability layer is then formed from a grid of overlapping grout bulbs to minimise groundwater inflow.

The combination of diaphragm walls and jet grout floor allows the main buried structure of the proposed sump station and associated conveyor tunnel to be formed in-situ. Water from within the excavation is then removed using either conventional well dewatering points or by pumping from a sump as the excavation of soil within the structure is excavated. The diaphragm walls and grout floor will form a low permeability structure limiting the volume of water requiring dewatering to the pore volume within the excavated soil plus a nominal ongoing minor seepage. A separate structural floor will be installed above the jet grout floor, resulting in a near impermeable final structure for the proposed 4^{th} dump station.

The conveyor tunnel is proposed to be constructed using a similar methodology, however may comprise either diaphragm walls or sheet pile walls with the floor seal formed using jet grouting. Sheet pile walls are formed by driving a series of interlocking steel sheets into the ground using a pile driving rig.

It is estimated that the volume of soil required to be removed from the main Dump Station and conveyor will be about 20,000 m^3 in total and an overall groundwater / pore water volume of about 4000 to 6000 m^3 (i.e. 4 to 6 ML of water).

2.3 Conveyor and Transfer House Footings

It is expected that either driven piles or continuous flight auger (CFA) piles will be required to support conveyor trestles as well as other structural elements including the transfer houses. The use of driven piles results in no excavation of soil, apart from shallow excavation for pile caps, which will occur to approximately 2 metre depth. The installation of CFA piles will involve the excavation of minor volumes of soil, within the order of 2 m^3 per pile to about 10 m depth.

No dewatering would be required for installation of piles, however in some instances the excavation for pile caps may involve intersection of shallow groundwater requiring minor sump and pump dewatering. This would be unlikely to occur under normal climatic conditions, however could occur if groundwater levels became elevated after prolonged wet weather.

3. DESCRIPTION OF SITE AND EXISTING FACILITIES

Kooragang Island is located on the lower reaches of the Hunter River and comprises an island about 10 km by 3 km, which has been formed by reclamation of a number of former smaller islands, channels and shallows. The Hunter River splits into the North and South Arms either side of the island.

PWCS activities are located on the south eastern portion of the island. The facilities allow unloading of coal brought in by rail and storage of the coal prior to transfer to ship loading facilities on the south arm of the Hunter River.

The existing dump stations are located side by side and comprise rail lines passing over underground concrete hoppers, covered by a portal frame structure. The dump stations have overall dimensions of approximately 50 m by 11m, with the floor of the hoppers at -10 m NHTG (see Section 5.2), however with diaphragm walls extending to a level of approximately -15 m NHTG at the eastern end of the dump stations for connection to conveyor tunnels.

The conveyor tunnels feed to existing transfer stations located approximately 230 m to the east, surfacing approximately 200 m from the dump stations, from where the conveyor continues above ground on trestles. The deeper sections of tunnels are of diaphragm wall construction with the walls extending from ground level to below the floor level of the tunnels. The tunnels are approximately 8 m wide with a height of approximately 4.5 m.

The surrounding ground at the Dump Station and Tunnels is relatively level in the range 5 m to 6 m NHTG.

4. REGIONAL GEOLOGY

The geology at the site comprises Permian aged Tomago Coal Measures overlain by Quaternary alluvium. The Tomago Coal Measures comprise shale, siltstone, sandstone, conglomerate and coal and is found at depths in the approximate range 45 m to 55 m at the site.

The overlying alluvium comprises fine grained estuarine sediments, overlain by fluvial sands with fine grained estuarine deposits at the top of the natural profile.

The site has been subsequently filled mostly with dredged sand, however there are also regions filled with blast furnace slag, clay fines and gypsum as part of the establishment of industrial land at Kooragang Island.

5. BACKGROUND

5.1 Subsurface Conditions

5.1.1 General

A generalised summary of the soil stratification and geotechnical units present at KCT is presented in Table 1 below. Specific subsurface details vary considerably across the site, and specific conditions at the dump station described in further detail in the following sections.

Table 1 - Summary of Subsurface Conditions and Geotechnical Units at KCT

Unit	Name	Description
1	Fill	Dredged fines, dredged sand and other granular fill to depths ranging from 2.6 m to 5.9 m, mainly comprising sand with some fines/clay. This layer forms the **Fill Aquifer**.
2	Alluvial Clay	Silty clay and clay, generally soft to firm where not pre-loaded, and up to stiff where previously loaded. The alluvial clay ranges in thickness up to 12 m across the site. This layer forms a confining layer or **Aquitard**, but is not always present.
3	Sand	Fine to medium grained sand with some shell fragments, generally medium dense then becoming dense to very dense. The sand extends to depths of about 30 m to 50 m across KCT. This layer forms the **Estuarine Aquifer**.
4	Estuarine Sediments	Stiff to very stiff estuarine clay and sandy clay, becoming hard in places, and underlain by various layers of clayey sand, gravelly sand and further clay layers, extending to bedrock.
5	Bedrock	Bedrock typically comprises siltstone and sandstone of the Tomago Coal Measures. The depth to rock varies across KCT from about 35 m to 80 m.

5.1.2 The Proposed Fourth Dump Station

The dump station site was the subject of a previous investigation by DP in 2000 (Ref 1) for the construction of the existing Dump Station 1.16 and Conveyor Tunnel 1.14. The investigation included the following:

- Review of previous data for the site (1982 to present);
- Two bores to depths of up to 20.4 m;
- Seven cone penetration tests to depths of up to 30 m;
- Acid sulphate soil sampling and testing;
- Installation of groundwater wells;
- In situ groundwater pumping test in the fill aquifer.

The results of the investigation indicated the following general subsurface profile within the vicinity of the proposed fourth dump station site.

Table 2- Subsurface Profile at Dump Station

Unit No	Description	RL (NHTG)		Thickness (m)
		Top of Unit	Base of Unit	
1	Fill - Silty Sand and Gravel	5.13 to 6.1	1.93 to 3.39	2.1 to 3.8
2	Clay and Sandy Clay	1.93 to 3.39	-0.45 to 0.91	1.4 to 3.0
3	Medium dense to dense, medium grained sand, some clayey sand and gravelly sand layers	-0.45 to 0.91	-30.8*	30*
4	Clay	-30.8*	NM	NM
5	Bedrock	NM	NM	NM

Notes to Table 2:

*Based on single bore about 200 m south of dump station
NM = Not Measured

Groundwater was measured at 2.75 m depth (RL 3.35 m) in the Fill Aquifer and at 4.5 m depth (RL 1.6 m) in Estuarine Aquifer at the time of the investigation (October 1999).

The estimated hydraulic conductivity of the aquifers were estimated from the results of pumping tests and particle size distribution and are presented in Table 3 below.

Table 3 - Summary of Estimated Aquifer Properties at Dump Station

	Estimated Range of Transmissivity (m^2/s)	Estimated Range of Hydraulic Conductivity (m/s)	Suggested Design Hydraulic Conductivity (m/s)
Upper Aquifer	NA	1×10^{-5} to 2.5×10^{-5}	2×10^{-5}
Lower Aquifer	7×10^{-3} to 13×10^{-3}	2.5×10^{-4} to 4.5×10^{-4}*	4×10^{-4}*

* Based on an estimated aquifer thickness of 30 m.

5.2 Groundwater Flow

5.2.1 Aquifers

The groundwater flow at the site primarily occurs within the two aquifers described in Section 5.1, which are separated by a ‘leaky’ clay aquitard. The aquifers and aquitard are discussed further below.

Contours of groundwater head in the Fill Aquifer and Estuarine Aquifers are provided for two times as follows:

- Drawings 13 and 14 (From Ref 2) provide contours of head in the Fill and Estuarine Aquifers. The contours are based on the most complete set of data for the overall site and surrounds, from measurements taken on the site in 2006 combined with data from wells on the NCIG site to the south of the overall site (about 500 m south-west of dump station site), measured in 2004. Groundwater levels are expected to have subsequently changed in the general vicinity of stone columns installed for Stage 3D of the PWCS site and wick drains on the NCIG site, due to penetration of the clay aquitard, however the this is not expected to have any significant influence at the dump station site;
- Drawings 1 and 2 attached present contours of head in the Fill and Estuarine Aquifers from well measurements taken on the PWCS site in 2008. The contours are based on a much more limited data set than for Drawings 13 and 14, however do represent more recent data.

5.2.2 Fill Aquifer (Unit 1 Sand Fill)

The Fill Aquifer is at the surface and is therefore unconfined. This means the water table fluctuates within the thickness of the aquifer, and groundwater is free to drain to the surface where the water table intersects the surface, such as at drains.

The Fill Aquifer is recharged primarily by rainfall. Groundwater flow within the fill is primarily sub-horizontal, generally flowing towards the closest surface drainage feature, however some vertical leakage occurs through the underlying clay aquitard, in particular where there are wick drains or stone columns. The nearest stone columns are about 300 m to the south of the dump station and would not affect local flow conditions at the dump station (the nearest wick drains are even further away).

Reference to Drawings 1 and 13 indicates that the groundwater in the Fill Aquifer in the vicinity of the dump station travels in a general northerly direction towards the tidal flats associated with the north arm of the Hunter River. In June 2008 the groundwater levels in the upper aquifer at the dump station site are interpolated to be at about RL 3.6 m which compares to about RL 3.5 in 2006 and RL 3.35 in Dec 1999. The variations are within the expected range of fluctuations which would be in response to variable rainfall conditions.

5.2.3 Clay Aquitard (Unit 2 Clay)

The Clay Aquitard primarily consists of Unit 2 clay (refer to Figure 2). The total thickness of the aquitard ranges from less than 1 m to over 5 m across the overall site and was measured in the range 1.4 m to 3.0 m thick in the vicinity of the dump station site. The permeability of the clay aquitard is low, however still sufficient to allow some vertical flow from the Fill Aquifer to the underlying Estuarine Aquifer.

5.2.4 Estuarine Aquifer (Unit 3 Sand)

The Estuarine Aquifer is confined, which means that there is no free water table within the layer, the potentiometric or phreatic surface (the height at which a water table would form in a bore connected only to the Estuarine Aquifer) is above the base of the overlying clay aquitard. The phreatic surface, is however below the water table in the Fill Aquifer, thereby allowing vertical flow from the Fill Aquifer to the underlying Estuarine Aquifer.

The sand is of moderate to high permeability. Reference to Drawings 2 and 14 indicates that the groundwater in the Estuarine Aquifer in the vicinity of the dump station travels in a general northerly direction towards north arm of the Hunter River. In June 2008 the groundwater levels in the lower aquifer at the dump station site are interpolated to be at about RL 2.2 m which compares to about RL 2.0 in 2006 and RL 1.6 in Dec 1999. The variations are within the expected range of fluctuations which would occur in response to variable rainfall conditions.

5.3 Groundwater Chemistry

Previous Dump Station Investigations 1999/2000

The groundwater chemistry at the dump station site was assessed in 1999 (Ref 1) prior to dewatering for the existing dump station three construction. These have been plotted against the trigger values for Slightly to Moderately Disturbed marine environments as per ANZECC (Ref 3) in Table 4 below.

Table 4 - Groundwater Chemistry December 1999

Parameter	I-C005 Estuarine Aquifer	DC-1 Duplicate of I-C005	I-C006 Fill Aquifer	Laboratory PQL	ANZECC Marine
pH	6.8	6.8	7.8	N/A	6.5-8.0
Alkalinity ($CaCO_3$/L)	310	300	160	N/A	NC
Turbidity (NTU)	43	76	730	N/A	1-50
Total Suspended Solids	99	140	790		NC
Anions					
Cl	550	680	360	0.5	NC
NO_3	<PQL	<PQL	<PQL	0.5	0.7
SO_4	210	230	220	0.5	NC
NH_3 (mg N/L)	1	<PQL	<PQL	1	0.9
Cations					
As	<PQL	<PQL	<PQL	0.005	NC
Cd	<PQL	<PQL	<PQL	0.005	0.0007
Ca	92	85	65	0.1	NC
Cr	0.005	<PQL	0.006	0.005	0.0044
Cu	<PQL	<PQL	0.006	0.005	0.0013
Fe	**2.3**	**3**	**17**	0.1	NC
Fe - filtrable	1	1.1	<PQL	0.1	NC
Pb	<PQL	<PQL	0.014	0.005	0.0044
Mg	34	31	10	0.1	NC
Hg	<PQL	<PQL	<PQL	0.001	0.0001
K	21	21	20	0.1	NC
Na	370	340	270	0.1	NC
Zn	0.042	0.038	0.1	0.01	0.015
Charge Balance Error	2.4	-9.6	0	NA	NA
TRH					
C_6 - C_9	<PQL	<PQL	<PQL	0.1	NC
C_{10} - C_{14}	4.4	4	0.3	0.1	NC
C_{15} - C_{28}	0.8	1	0.5	0.1	NC
C_{29} - C_{36}	<PQL	0.2	0.4	0.1	NC
Total TRH					0.0007
BTEX					
Benzene	<PQL	<PQL	<PQL	0.001	0.5
Toluene	<PQL	<PQL	<PQL	0.001	0.18
Ethyl Benzene	<PQL	<PQL	<PQL	0.001	0.08
Xylene	<PQL	<PQL	<PQL	0.001	NC
Total PAHs	<PQL	<PQL	<PQL	0.02	NC
Grease and Oil	<PQL	<PQL	<PQL	5	NC
Total Phenolics	<PQL	<PQL	<PQL	0.04	0.4
Cyanide	0.01	0.01	0.03	0.01	0.004

Notes to Table 4:
All Results in mg/l PQL = Practical Quantification Limit
Bold entries indicate exceedance of EPA licence conditions (if extracted and discharged)
Shaded entries indicated exceedance of ANZECC Slightly to Moderately Disturbed marine environments criteria.

It is understood that disposal of the extracted groundwater was managed by transferring the water to the sediment ponds for treating prior to disposal to the north arm of the Hunter River. The treatment comprised flocculation of the iron. No pH adjustment was required.

It is noted however that a number of additional parameters exceed the current ANZECC marine guidelines. These parameters include the following:

- ***Fill Aquifer***: Chromium, copper, Lead, Zinc, Cyanide, Total Recoverable Hydrocarbons (TRH);
- ***Estuarine Aquifer***: Chromium, Zinc, Cyanide, TRH.

DP Groundwater Review 2009

In 2009 DP undertook a review of the results of previous sampling and testing of groundwater on the KCT site. The information available for the review comprised three annual monitoring reports by RCA Australia for 2001, 2002 and 2008, plus seven reports by GHD-Longmac describing the installation and monitoring of various wells throughout KCT.

The chemical test results for each parameter, plus the recorded water levels, have been plotted against time. The results for the two aquifers are plotted separately for ease of distinction. The graphs are attached. Review of these graphs enables a visual review of concentrations levels, ranges and trends for individual wells and groups of wells. Most parameters are plotted on a logarithmic scale due to the wide variation in values.

The well locations referred to on the graphs are shown on Drawings 1 and 2 attached.

The graphs also show the relevant ANZECC 2000 criteria (Ref 3), taken as the trigger value for 95% protection, for slightly to moderately disturbed ecosystems in a marine environment (red horizontal line). Where marine water criterion is not given in ANZECC the corresponding fresh water criterion was used. Some parameters have no ANZECC criteria at all, and no line is plotted.

The following observations are made regarding groundwater levels and chemistry trends across the overall PWCS site:

Upper (Fill) Aquifer:

- Water levels have generally remained steady at individual wells, although the level difference between wells EH32W-U and EH33W-U had reversed;
- pH levels indicate a slight decline between 2002 and 2008, however there are no intervening readings to confirm a trend;
- Most metals at most wells are *usually* below the relevant ANZECC criteria, except Iron, copper and zinc. Occasional spikes above ANZECC have occurred with arsenic, chromium, lead, manganese, mercury, nickel and selenium;
- In the 2008 rounds of sampling cyanide, ammonia and total phenolics have exceeded ANZECC criteria in some of the wells;
- Some of the fuel farm tank recorded exceedances of TPH, naphthalene and phenanthrene and benzo(a)pyrene (the latter three being PAHs however such results were only recorded locally near the fuel tanks.

Lower (Estuarine) Aquifer:

- The water level in the lower aquifer has remained generally consistent over the period of record;
- pH values are reasonably consistent and similar to the Upper Aquifer;
- The Lower Aquifer is more saline than the Upper Aquifer, based on Electrical Conductivity (EC);
- Most metals at most wells are *usually* below the relevant ANZECC criteria, except copper and zinc. Occasional spikes above ANZECC have occurred with arsenic, cadmium, chromium, manganese, mercury, nickel and selenium during the sampling periods;
- In the 2008 sampling rounds ammonia exceeded ANZECC criteria in most of the wells;

- Most of the well locations subject to recent testing are located remote from the Dump Station site, with the exception of EH29W in the Fill Aquifer, located about 150 m to the north. Data was available from the 2008 monitoring and indicated that the following parameters exceeding the ANZECC criteria in the Fill Aquifer: Copper, Nickel, Zinc, and Ammonia. The results of TRH and Anthracrene were below the Practical Quantitation Limits (PQLs) for the testing undertaken, however these were above the ANZECC criteria and therefore it is possible, although unlikely, that exceedances occurred.

5.4 Acid Sulphate Soils

Testing for acid sulphate soils was undertaken in the vicinity of the dump station site DP in 1999 (Ref 1). The results of the testing indicated the following:

- The results of pH in KCl and Total Actual Acidity (TAA) confirmed that there were no actual acid sulphate soils present;
- The results of laboratory testing indicated that the fill aquifer filling did not contain actual or potential acid sulphate soils;
- The results of POCAS testing indicated that the samples of clay aquitard exceeded the action criteria;
- The Estuarine Aquifer (Unit 3) samples at 5.25 m and 6.75 m exceeded criteria for both the acid and sulphur trails. The remaining samples from 9.75 m depth and deeper did not exceed either action criteria.

Based on this analysis the soils associated with clay aquitard (Unit 2) and the upper levels of the estuarine aquifer (Unit 3) are identified as potential acid sulphate soils (PASS).

An acid sulphate soil management plan was developed for the following soil units:

1. All Unit 2 soil (clay) – clay aquitard between the fill and estuarine aquifers.
2. Unit 3 soil (sand) to a depth of 10 m (RL -4 NHTG).

Based on this analysis, it was considered that all Unit 1 material as well as Unit 3 material below 10 m depth did not require management and could be excavated and utilised for general filling purposes on site without treatment.

The acid sulphate soil management plan recommended the following general management procedures:

- Transfer all Unit 2 and Unit 3 soils to 10 m depth to Fines Disposal Facility;
- Sample soil and confirm it requires treatment;
- Treat soil by lime neutralisation;
- Verify treatment by additional soil sampling and testing;
- Contain leachate and lime treat, as required, prior to offsite disposal.

It is however understood that the FDF is no longer available for treatment and disposal of acid sulphate soils. The potential acid sulphate soils will therefore require treatment close to the excavation site, then either re-used on site or disposed to a licenced landfill as *General Solid Waste* (subject to meeting all other criteria for *General Solid Waste*). Section 6.3.2 discusses this further.

The potential for the drawing down of the water table, leading to oxidation of in-situ soils was identified, however it was considered that the extent of dewatering would be localised and the pH would be buffered by mixing with the surrounding unaffected water.

5.5 Previous Dewatering

The dewatering system used for construction of dump station three was designed by Environmental & Groundwater Management (EGM), as outlined in its report of July 2000 (Ref 5).

The EGM report indicated the following:

- Dewatering would be undertaken in two stages, Stage 1 comprising dewatering of the Dump Station to RL -4.8 m to allow construction of the top floor of the dump station. Fifteen wells were to be installed and the projected discharge rate was 155.8 L/s (13,461 m^3/day);
- Stage 2 of the dewatering was divided into two phases. Phase 1 comprised dewatering the Dump Station to RL -11.7 and Phase 2 comprised dewatering of the Tunnel to RL -8.7 m. The combined flow rate for these two stages was 267.6 l/s (23,077 m^3/day)using 18 wells;
- The wells were to be installed to 36 m depth, of 200 mm diameter and would be screened across both the Fill and Estuarine Aquifers. Airlift pumps were to be installed in each well;
- The extent of drawdown was predicted for both stages of dewatering and indicated the following for the Estuarine Aquifer:
  - Stage 1 after 20 days:
    - 100 m distance: RL -4.1 (7.45 m drawdown);
    - 800 m distance: RL 1.0 (2.5 m drawdown).
  - Stage 2 after 30 days:
    - 100 m distance: RL -7 (10.35 m drawdown);
    - 800 m distance: RL 0.5 m (2.85 m drawdown).
- The extent of drawdown in the Fill Aquifer was predicted to be very localised, with effects limited to less than 20 m from the dump station;
- The potential for ground settlement associated with the dewatering was commented on, but no specific estimates were provided;
- An option was provided for recharging some of the extracted groundwater to limit potential settlement. The option was for a line of 15 wells at 10 m centres, the exact location of which was not nominated. Based on a flow rate of 5.7 l/s per well this would raise/restore the draw down water about 6 m near the line of wells and 1 m at 200 m distance. For 15 wells this flow rate would be about one third of the Stage 2 pumping rate.

No results of monitoring of the actual drawdowns which occurred during dewatering were available for this report.

6. COMMENTS

6.1 General

The components of the proposed development which could have potential to impact on groundwater would be those that involve elements extending below the water table. This would include the following:

- Dump station and subsurface section of conveyor;
- Footings for proposed conveyor and transfer houses.

Conventional dewatering, as undertaken for construction for previous dump stations, has been considered, however is not proposed for the new dump station as the settlement ponds are no longer available for treatment of the water prior to discharge. The option of conventional dewatering with re-injection of the extracted water has also been also considered, however is not proposed due to the potential high flow rates associated with full re-injection of the water. It was considered that re-injection would be more practical if the flow rates could be reduced by sealing the walls and base of the excavation.

As outlined in Section 2.2, the proposed construction methodology of the dump station, comprising diaphragm / sheet pile walls and jet grouting of the floor structure will result in a low permeability structure which can be constructed within minimal dewatering. One-off dewatering of the pore water from within the structure will be required, as well as some minor ongoing seepage.

It is possible that a similar result to jet grouting could be achieved by sealing the diaphragm wall into the underlying low permeability stiff estuarine clay (Unit 4), which is known to be generally present elsewhere on the KCT site. The efficacy of this option, however, would be dependent on the actual depth of the estuarine clay, which has not yet been determined beneath the fourth dump station site.

The construction process for the dump station will require excavation of acid sulphate soils from within the structure. This will require an appropriate management strategy, as discussed in following sections.

Piled footings will be required along the conveyor and for structures including transfer house and associated coal handling infrastructure. The piled footings will not require dewatering. If CFA piles are used then there is the potential, for disturbance of acid sulphate soils which will required appropriate management. There is also a low risk of localised dewatering associated with construction of shallow pile caps.

6.2 Dump Station and Conveyor

6.2.1 Design and Construction

The volume of soil contained in the proposed dump station excavation is approximately 10,500 m^3. The volume of soil within the associated conveyor tunnel is expected to be a further approximately 10,000 m^3 (total of 20,500 m^3). Therefore, based on a specific yield in the range 0.2 to 0.3 the volume of pore water contained in this soil will be in the order of 4000 m^3 to 6000 m^3 (i.e. 4 to 6 ML).

Once the dump station/conveyor walls and floor are completed the water contained within the structure will be dewatered either by installing wells, or by pumping from localised sumps as the soil is excavated.

There will be potential for some ongoing seepage into the structure, the rate of which can be minimised by careful attention to design, construction and verification, in particular with respect to construction of the jet grouted floor.

The following measures should be undertaken to reduce the risk of excessive leakage of the structure:

- Selection of an experienced contractor with a proven track record of installing successful jet grouting floor structures;
- Design the jet grouted structure to withstand uplift forces. This could include one or a combination of the following measures:
  - Extend floor across/below base of diaphragm walls to transfer uplift to walls;
  - Adopt thickness of floor to provide sufficient strength to resist bending due upwards pressure on base of wall;
  - Install ground anchors through floor into underlying soil;
  - Set the jet grout floor deeper than the final dump station floor to allow a mass of soil to remain over the jet grout floor to hold it down. A separate floor would be required to be constructed above the remnant soil mass.
- Adoption of an appropriate grid spacing to suit the proposed equipment and soil conditions and reduce the risk of gaps between the grout plugs;
- Careful monitoring of jet grouting process, including jet pressures and grout takes;
- Hydraulic testing of completed structure by undertaking pumping tests to assess for leakage;
- Coring of the grouted floor to check consistency of thickness and strength of grouted floor;
- Application of secondary grouting if considered necessary from the results of construction monitoring, pumping tests and coring;
- Review of the design as well as monitoring of construction should be undertaken by a third party specialist.

The proposed construction techniques have been demonstrated to be successful in a range of construction projects in a range of environments. For example, it is understood that jet grout seals were used for sealing the base of 15 m base of diameter shafts at 15 m depth below the water table for the Sydney Desalination Project. The seals were 4 m to 6 m thick and one of the three seals required secondary grouting following hydraulic testing. Also attached is a project summary of another similar project undertaken in Florida, demonstrating the success of the proposed design technique.

For the potential design option case that the diaphragm wall is sealed into low permeability clay then a similar process would be required, in order to confirm the consistency and permeability of the clay.

6.2.2 Predicted In-flow Rates

Some preliminary calculations have been undertaken to assess potential seepage rates through the jet grouted floor. Kutzner, "Grouting of Soil and Rock", suggests a typical permeability for constructed cement jet grouting in cohesive soils is less than 10^{-8} m/s, similar to a clay soil. Based on an upper bound permeability of 10^{-8} m/s and a two metre thick jet grout floor, the expected flow rate through the floor of the dump station would be about 3.5 m^3 (or 0.0035 ML per day) per day. If the permeability of the floor were to be an order of magnitude higher, which is less likely, then the flow rate would be about 35 m^3/day (or 0.035 ML per day) which is still many orders of magnitude less than for conventional external dewatering.

If the conveyor tunnel were formed using diaphragm walls, then similar or lesser flow rate would be expected resulting in a total flow of approximately 7 m^3 to 10 m^3 per day (0.007 ML to 0.01 ML per day). If sheet pile walls were used for the construction, with formed concrete walls constructed for the completed structure then higher flows could be expected through the sheet piles walls during the construction phase. The flow rates would depend on the type of sheet pile walls however would easily be an order of magnitude higher than for the diaphragm wall option. The suitability of sheet pile walls should be confirmed by detailed design.

It is noted that higher flow rates could occur if the jet grouting was deficient and this highlights the need for appropriate design, construction and verification which would substantially reduce the risk of increased flow rates.

6.2.3 Potential Impacts on Groundwater

A number of potential impacts which may be typically expected when undertaking conventional dewatering have been considered, however the proposed construction technique substantially reduces or removes these impacts, as discussed in the following sections.

Drawdowns

The predicted seepage rate into the sealed excavation is predicted to be approximately 7 m^3 to 10 m^3 per day (or 0.007 ML to 0.01 ML per day), many order of magnitude less than predicted for the conventional dewatering. As a consequence, drawdown of groundwater outside the structure is expected to be insignificant and less than seasonal variations in water table level.

Flow Rates and GDEs

The low seepage rates and drawdowns will result in little disruption to the base groundwater flow rate towards the north arm of the Hunter River.

This means that the construction is not expected to affect the groundwater-dependant ecosystems to the north of the site. Similarly it is considered that saltwater intrusion, which can sometimes occur when dewatering close to a saline water body, is not an issue.

De-Saturation and Acid Sulphate Soils

Although minimal drawdowns are expected outside the sealed structure, the soil inside the structure will be de-saturated, aerating the clay aquitard (Unit 2) and the upper parts of the Estuarine Aquifer (Unit 3), which have been identified as potential acid sulphate soils (PASS). This could possibly lead to oxidation of the PASS and generation of acidic groundwater conditions. The water will be fully contained within the sealed structure and therefore will not impact on the surrounding groundwater.

The water, however, may require neutralisation prior to re-injection.

Potential Subsidence

Ground subsidence is often a potential issue when dewatering, due to the drawn-down water levels increasing the effective stress and inducing subsidence. As the drawdown external to the dump station is expected to be insignificant, subsidence is not expected to be an issue.

Existing Water Quality

Based on results and analysis of groundwater quality, it is expected that the main chemical parameter of concern for groundwater disposal will be Iron. Chemical monitoring of groundwater across the site, however also indicates that potential presence of contaminants exceeding the ANZECC 2000 guidelines for marine ecosystems, including the following:

- Arsenic is occasionally found at concentrations above the criteria, however in the 2008 round of testing the only exceedance on the overall site were in the Fill Aquifer at locations FF5 (fuel farm) and EH31 (1.4 km to west);
- Copper and Zinc concentrations regularly exceed the criteria at most locations across the site, including at EH29 (250 m east of site) for the 2008 round of testing. The presence of copper and zinc at concentrations above the ANZECC criteria is typical for most groundwater in the region and the concentrations are expected to be similar to background concentrations;
- Cadmium, Chromium, Lead and Manganese concentrations often exceeded the criteria for testing undertaken in the period 1996 to 2002, including the sampling and testing undertaken for the Fill Aquifer in previous dump station dewatering in 1999. However the results of more recent testing in 2008 indicated no exceedances of the criteria;
- Mercury concentrations exceeded the criterion at a number of locations in the 2008 round of testing. The results at EH29W, which is closest to the dump station site, indicated that the concentration in the Fill Aquifer was the same as the criterion;
- A scatter of results have been recorded for Selenium, some of which have exceeded the criterion. In the 2008 round the only exceedances were at EH27W in the Fill Aquifer (700 m to NW) and EH27, EH32 and EH33 in the Estuarine Aquifer (all greater than 700 m to the west);

- Nickel has historically had exceedances in both aquifers, however in 2008 there we no exceedances in the Estuarine Aquifer. Exceedances in the Fill Aquifer occurred at several locations including EH29 (250 m east of site);
- A number of exceedances of Arsenic were recorded in 1999 to 2001. including testing at the Dump Station in 1999, however no exceedances were recorded in 2008;
- The concentrations of TRH are generally less than the PQL, however has generally been above the ANZECC criteria. Some high concentrations were recorded in both aquifers at the Dump Station in 1999, however these are not repeated in any of the other testing and are likely to have been a localised occurrence.

Therefore the water would generally not be suitable for disposal to surface water without treatment, however re-injection of the water to the estuarine aquifer would not be expected to have adverse impacts, as discussed in more detail in Section 6.3.1.

6.3 Mitigation of Groundwater Impacts from Dump Station Dewatering

The issues of drawdown, changes to groundwater flow rates and salt water intrusion are not expected to produce negative impacts and therefore no mitigations have been proposed.

6.3.1 Disposal of Water

General

The volumes of water expected to be extracted from the Dump Station and conveyor are in the order of an initial 4000 m^3 to 6000 m^3 (4 ML to 6 ML) with an ongoing flow rate of about 10 m^3/day 0.01 ML per day) until final sealing of the structure is complete, following which the flow rates will be very low.

It is considered that the water produced can be managed by either of the following:

- re-injection of the water into the Estuarine Aquifer, with minimal treatment; or
- on-site treatment prior to re-injection and/or reuse on site through the existing KCT water management system.

These options are discussed further below.

Re-Injection

Based on the expected volumes of water requiring re-injection it is expected that one to three wells installed into the Estuarine Aquifer would be sufficient for re-injection and would be expected to lead to insignificant mounding of the water table.

The quality of the groundwater at the specific dump station site should be verified prior to re-injection to ensure that the background groundwater quality will not be affected. Based on existing results to date, the reinjected water would be of similar quality to the background water quality. The process of re-injection will likely lead to aeration of the water which may actually attenuate the concentrations of some potentially present contaminants, most particularly organics such as TRH.

The re-injection wells would comprising casing installed through the upper fill and clay layers to the natural sand that forms the Estuarine Aquifer. Suitable locations for the reinjection wells would include undeveloped areas close to the dump station site, such as the strip of land south-west of the dump station between the access road and rail line. Locations within the rail loop, north-east of the dump station could also be considered, provided that a suitable pipeline crossing over the rail lines can be achieved (e.g. over the rail bridge or through a culvert).

De-saturation of the soil will occur within the excavation. The re-injection system could easily accommodate appropriate lime treatment of the water prior to re-injection to neutralise any acidity generated from acid sulphate soils.

On-Site Treatment

The water quality at the specific dump station site may require treatment prior to re-injection, or reuse on site through the existing KCT water management system. This would involve mobilisation of a specialised treatment plant for on-site treatment .

Information from remediation contractors indicates that treatment of the expected water quality and quantity to meet ANZECC slightly disturbed marine criteria is readily achievable using mobile plant. Various size mobile plants with flow capacities in the range 4m^3/hr to 17m^3/hr (408 m^3/day) are commercially available for hire. For the larger plant a period of about two weeks would be required to treat the initial volume of water extracted.

Depending on the identified contaminants requiring treatment the plant would be expected to include the following processes:

- pH adjustment for treatment of metals;
- Ferric bed for cyanide/ammonia;
- Carbon filter for TRH.

This equipment has been used with success on industrial sites in the region, with comparatively worse groundwater quality. It is understood that the salinity of the water will not adversely affect the performance of the plant.

6.3.2 Acid Sulphate Soils

An acid sulphate soils management plan has been developed for excavation and treatment of soil and for dewatering (Ref 6). This plan includes the following components:

- Transfer to treatment area;
- Neutralisation with lime;
- Verification testing and monitoring.

6.4 Impacts and Mitigations for Conveyor and Wharf

The construction of the conveyors, wharf and other structures are expected have relatively minor impacts on groundwater. It is expected that occasional shallow and localised excavations will be required for installation of pile caps. These are unlikely to require dewatering and if dewatering was required then it would be localised and short term and only affect the Fill Aquifer. Any dewatering associated with the pile cap construction would be accommodated in the existing water management system at KCT.

The installation of piles and pile caps will require excavation of soil and this will be undertaken in accordance with an acid sulphate soil management plan where acid sulphate soils are penetrated (clay aquitard and upper parts of estuarine aquifer).

7. SUMMARY

In summary, a desktop assessment has been undertaken with respect to existing groundwater flows, groundwater quality and the presence of acid sulphate soils. The desktop study has been undertaken to identify possible adverse impacts on water quality due to proposed the proposed Stage 4 Project, and proposed suitable measures to mitigate the impacts.

The construction methodology of the proposed 4^{th} Dump Station , comprising the use of diaphragm walls with a jet grouted floor (or with the walls installed to the deep clay layer), has been adopted to avoid impacts which could otherwise occur with conventional dewatering. Drawdown of the surrounding aquifers and associated issues such as desaturation of Groundwater Dependant Ecosystems, extensive disturbance of acid sulphate soils, changed flow directions and salt water intrusion are all prevented by the proposed construction methodology.

Acid sulphate soils will be present within the excavation, which can be readily managed by adopting an appropriate acid sulphate soils management plan (ASSMP), as attached. The water removed from the excavation will be of limited volume and of similar quality to the existing water and therefore is expected to be suitable for re-injection. As a contingency, it would be practical to install a mobile/temporary groundwater treatment plant to treat the water extracted during construction, to a quality suitable for either re-injection or reuse on site. A licence for dewatering will be required from DECCW.

The installation of bored (CFA) piles associated with conveyors and other structure will disturb minor amounts of acid sulphate soils and these will need to be managed in accordance with the ASSMP.

8. LIMITATIONS

Conditions on site different to those identified during this assessment may exist. Therefore DP cannot provide unqualified warranties nor does DP assume any liability for site conditions not recorded in the data available for this study.

This report and associated documentation and the information herein have been prepared solely for the use of Umwelt and PWCS. Any reliance assumed by other parties on this report shall be at such party's own risk. Any ensuing liability resulting from use of the report by other parties cannot be transferred to DP.

Douglas Partners (DP) has prepared this report Umwelt and PWCS for this project at PWCS KCT in accordance with DP's proposal dated 27 July 2009 and acceptance received from Umwelt dated 28 July 2009. The work was carried out under DPs Conditions of Engagement in tandem with Umwelt Subconsultant Conditions of Engagement as amended by DP on 29 July 2009. This report is provided for the exclusive use of the Umwelt and PWCS for the specific project and purpose as described in the report. It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party.

The results provided in the report are considered to be indicative of the sub-surface conditions on the site only to the depths investigated at the specific sampling and/or testing locations, and only at the time the work was carried out. DP's advice may be based on observations, measurements, tests or derived interpretations. The accuracy of the advice provided by DP in this report is limited by unobserved features and variations in ground conditions across the site in areas between test locations and beyond the site boundaries or by variations with time. The advice may be limited by restrictions in the sampling and testing which was able to be carried out, as well as by the amount of data that could be collected given the project and site constraints. Actual ground conditions and materials behaviour observed or inferred at the test locations may differ from those which may be encountered elsewhere on the site. Should variations in subsurface conditions be encountered, then additional advice should be sought from DP and, if required, amendments made.

This report must be read in conjunction with the attached "Notes Relating to This Report" and any other attached explanatory notes and should be kept in its entirety without separation of individual pages or sections. DP cannot be held responsible for interpretations or conclusions from review by others of this report or test data, which are not otherwise supported by an expressed statement, interpretation, outcome or conclusion stated in this report. In preparing this report DP has necessarily relied upon information provided by the client and/or their agents.

DOUGLAS PARTNERS PTY LTD

Reviewed by:

Will Wright
Principal

Stephen Jones
Principal

REFERENCES

1. Douglas Partners, “Report on Dump Station and Conveyor Tunnel, PWCS Stage 3 Expansion, Kooragang Coal Loader”, 31100C, March 2000.

2. Douglas Partners, “Review of Groundwater Monitoring, Carrington Coal Terminal, Kooragang Coal Terminal & Fines Disposal Facility” Project 49322, April 2009.

3. ANZECC (2000), “Australian and New Zealand Guidleines for Fresh and Marine Water Quality”, October 2000.

4. Douglas Partners, “Report on Acid Sulphate Soil Management Plan, Dump Station and Conveyor Tunnel, PWCS Stage 3 Expansion, Kooragang Coal Loader”, 31100C15, February 2000.

5. Environmental and Groundwater Management, “ Redesigned Dewatering System for Port Waratah Coal Services, Stage 3 Expansion, Kooragang”, Job No 24284, July 2000

6. Douglas Partners “Acid Sulphate Soil Management Plant, Stage 4 Project, PWCS Kooragang Coal Terminal”, Project 49425-ASSMP, October 2009.

NOTES RELATING TO THIS REPORT

Introduction

These notes have been provided to amplify the geotechnical report in regard to classification methods, specialist field procedures and certain matters relating to the Discussion and Comments section. Not all, of course, are necessarily relevant to all reports.

Geotechnical reports are based on information gained from limited subsurface test boring and sampling, supplemented by knowledge of local geology and experience. For this reason, they must be regarded as interpretive rather than factual documents, limited to some extent by the scope of information on which they rely.

Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726, Geotechnical Site Investigations Code. In general, descriptions cover the following properties - strength or density, colour, structure, soil or rock type and inclusions.

Soil types are described according to the predominating particle size, qualified by the grading of other particles present (eg. sandy clay) on the following bases:

Soil Classification	Particle Size
Clay	less than 0.002 mm
Silt	0.002 to 0.06 mm
Sand	0.06 to 2.00 mm
Gravel	2.00 to 60.00 mm

Cohesive soils are classified on the basis of strength either by laboratory testing or engineering examination. The strength terms are defined as follows.

Classification	Undrained Shear Strength kPa
Very soft	less than 12
Soft	12—25
Firm	25—50
Stiff	50—100
Very stiff	100—200
Hard	Greater than 200

Non-cohesive soils are classified on the basis of relative density, generally from the results of standard penetration tests (SPT) or Dutch cone penetrometer tests (CPT) as below:

Relative Density	SPT "N" Value (blows/300 mm)	CPT Cone Value (q_c — MPa)
Very loose	less than 5	less than 2
Loose	5—10	2—5
Medium dense	10—30	5—15
Dense	30—50	15—25
Very dense	greater than 50	greater than 25

Rock types are classified by their geological names. Where relevant, further information regarding rock classification is given on the following sheet.

Sampling

Sampling is carried out during drilling to allow engineering examination (and laboratory testing where required) of the soil or rock.

Disturbed samples taken during drilling provide information on colour, type, inclusions and, depending upon the degree of disturbance, some information on strength and structure.

Undisturbed samples are taken by pushing a thin-walled sample tube into the soil and withdrawing with a sample of the soil in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Details of the type and method of sampling are given in the report.

Drilling Methods.

The following is a brief summary of drilling methods currently adopted by the Company and some comments on their use and application.

Test Pits — these are excavated with a backhoe or a tracked excavator, allowing close examination of the in-situ soils if it is safe to descent into the pit. The depth of penetration is limited to about 3 m for a backhoe and up to 6 m for an excavator. A potential disadvantage is the disturbance caused by the excavation.

Large Diameter Auger (eg. Pengo) — the hole is advanced by a rotating plate or short spiral auger, generally 300 mm or larger in diameter. The cuttings are returned to the surface at intervals (generally of not more than 0.5 m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube sampling.

Continuous Sample Drilling — the hole is advanced by pushing a 100 mm diameter socket into the ground and withdrawing it at intervals to extrude the sample. This is the most reliable method of drilling in soils, since moisture content is unchanged and soil structure, strength, etc. is only marginally affected.

Continuous Spiral Flight Augers — the hole is advanced using 90—115 mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or in-situ testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are very disturbed and may be contaminated. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively lower reliability, due to remoulding, contamination or softening of samples by ground water.

Non-core Rotary Drilling — the hole is advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from 'feel' and rate of penetration.

Rotary Mud Drilling — similar to rotary drilling, but using drilling mud as a circulating fluid. The mud tends to mask the cuttings and reliable identification is again only possible from separate intact sampling (eg. from SPT).

Continuous Core Drilling — a continuous core sample is obtained using a diamond-tipped core barrel, usually 50 mm internal diameter. Provided full core recovery is achieved (which is not always possible in very weak rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation.

Standard Penetration Tests

Standard penetration tests (abbreviated as SPT) are used mainly in non-cohesive soils, but occasionally also in cohesive soils as a means of determining density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, "Methods of Testing Soils for Engineering Purposes" — Test 6.3.1.

The test is carried out in a borehole by driving a 50 mm diameter split sample tube under the impact of a 63 kg hammer with a free fall of 760 mm. It is normal for the tube to be driven in three successive 150 mm increments and the 'N' value is taken as the number of blows for the last 300 mm. In dense sands, very hard clays or weak rock, the full 450 mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form.

- In the case where full penetration is obtained with successive blow counts for each 150 mm of say 4, 6 and 7

  as 4, 6, 7
  N = 13

- In the case where the test is discontinued short of full penetration, say after 15 blows for the first 150 mm and 30 blows for the next 40 mm

  as 15, 30/40 mm.

The results of the tests can be related empirically to the engineering properties of the soil.

Occasionally, the test method is used to obtain samples in 50 mm diameter thin walled sample tubes in clays. In such circumstances, the test results are shown on the borelogs in brackets.

Cone Penetrometer Testing and Interpretation

Cone penetrometer testing (sometimes referred to as Dutch cone — abbreviated as CPT) described in this report has been carried out using an electrical friction cone penetrometer. The test is described in Australian Standard 1289, Test 6.4.1.

In the tests, a 35 mm diameter rod with a cone-tipped end is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the friction resistance on a separate 130 mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are connected by electrical wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck.

As penetration occurs (at a rate of approximately 20 mm per second) the information is plotted on a computer screen and at the end of the test is stored on the computer for later plotting of the results.

The information provided on the plotted results comprises: —

- Cone resistance — the actual end bearing force divided by the cross sectional area of the cone — expressed in MPa.
- Sleeve friction — the frictional force on the sleeve divided by the surface area — expressed in kPa.
- Friction ratio — the ratio of sleeve friction to cone resistance, expressed in percent.

There are two scales available for measurement of cone resistance. The lower scale (0—5 MPa) is used in very soft soils where increased sensitivity is required and is shown in the graphs as a dotted line. The main scale (0—50 MPa) is less sensitive and is shown as a full line.

The ratios of the sleeve friction to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1%—2% are commonly encountered in sands and very soft clays rising to 4%—10% in stiff clays.

In sands, the relationship between cone resistance and SPT value is commonly in the range:—

$$q_c \text{ (MPa)} = (0.4 \text{ to } 0.6) \text{ N (blows per 300 mm)}$$

In clays, the relationship between undrained shear strength and cone resistance is commonly in the range:—

$$q_c = (12 \text{ to } 18)\ c_u$$

Interpretation of CPT values can also be made to allow estimation of modulus or compressibility values to allow calculation of foundation settlements.

Inferred stratification as shown on the attached reports is assessed from the cone and friction traces and from experience and information from nearby boreholes, etc. This information is presented for general guidance, but must be regarded as being to some extent interpretive. The test method provides a continuous profile of engineering properties, and where precise information on soil classification is required, direct drilling and sampling may be preferable.

Hand Penetrometers

Hand penetrometer tests are carried out by driving a rod into the ground with a falling weight hammer and measuring the blows for successive 150 mm increments of penetration. Normally, there is a depth limitation of 1.2 m but this may be extended in certain conditions by the use of extension rods.

Two relatively similar tests are used.

- Perth sand penetrometer — a 16 mm diameter flat-ended rod is driven with a 9 kg hammer, dropping 600 mm (AS 1289, Test 6.3.3). This test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.
- Cone penetrometer (sometimes known as the Scala Penetrometer) — a 16 mm rod with a 20 mm diameter cone end is driven with a 9 kg hammer dropping 510 mm (AS 1289, Test 6.3.2). The test was developed initially for pavement subgrade investigations, and published correlations of the test results with California bearing ratio have been published by various Road Authorities.

Laboratory Testing

Laboratory testing is carried out in accordance with Australian Standard 1289 "Methods of Testing Soil for Engineering Purposes". Details of the test procedure used are given on the individual report forms.

Bore Logs

The bore logs presented herein are an engineering and/or geological interpretation of the subsurface conditions, and their reliability will depend to some extent on frequency of sampling and the method of drilling. Ideally, continuous undisturbed sampling or core drilling will provide the most reliable assessment, but this is not always practicable, or possible to justify on economic grounds. In any case, the boreholes represent only a very small sample of the total subsurface profile.

Interpretation of the information and its application to design and construction should therefore take into account the spacing of boreholes, the frequency of sampling and the possibility of other than 'straight line' variations between the boreholes.

Ground Water

Where ground water levels are measured in boreholes, there are several potential problems;

- In low permeability soils, ground water although present, may enter the hole slowly or perhaps not at all during the time it is left open.
- A localised perched water table may lead to an erroneous indication of the true water table.
- Water table levels will vary from time to time with seasons or recent weather changes. They may not be the same at the time of construction as are indicated in the report.
- The use of water or mud as a drilling fluid will mask any ground water inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water observations are to be made.

More reliable measurements can be made by installing standpipes which are read at intervals over several days, or perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from a perched water table.

Engineering Reports

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg. a three storey building), the information and interpretation may not be relevant if the design proposal is changed (eg. to a twenty storey building). If this happens, the Company will be pleased to review the report and the sufficiency of the investigation work.

Every care is taken with the report as it relates to interpretation of subsurface condition, discussion of geotechnical aspects and recommendations or suggestions for design and construction. However, the Company cannot always anticipate or assume responsibility for:

- unexpected variations in ground conditions — the potential for this will depend partly on bore spacing and sampling frequency
- changes in policy or interpretation of policy by statutory authorities
- the actions of contractors responding to commercial pressures.

If these occur, the Company will be pleased to assist with investigation or advice to resolve the matter.

Site Anomalies

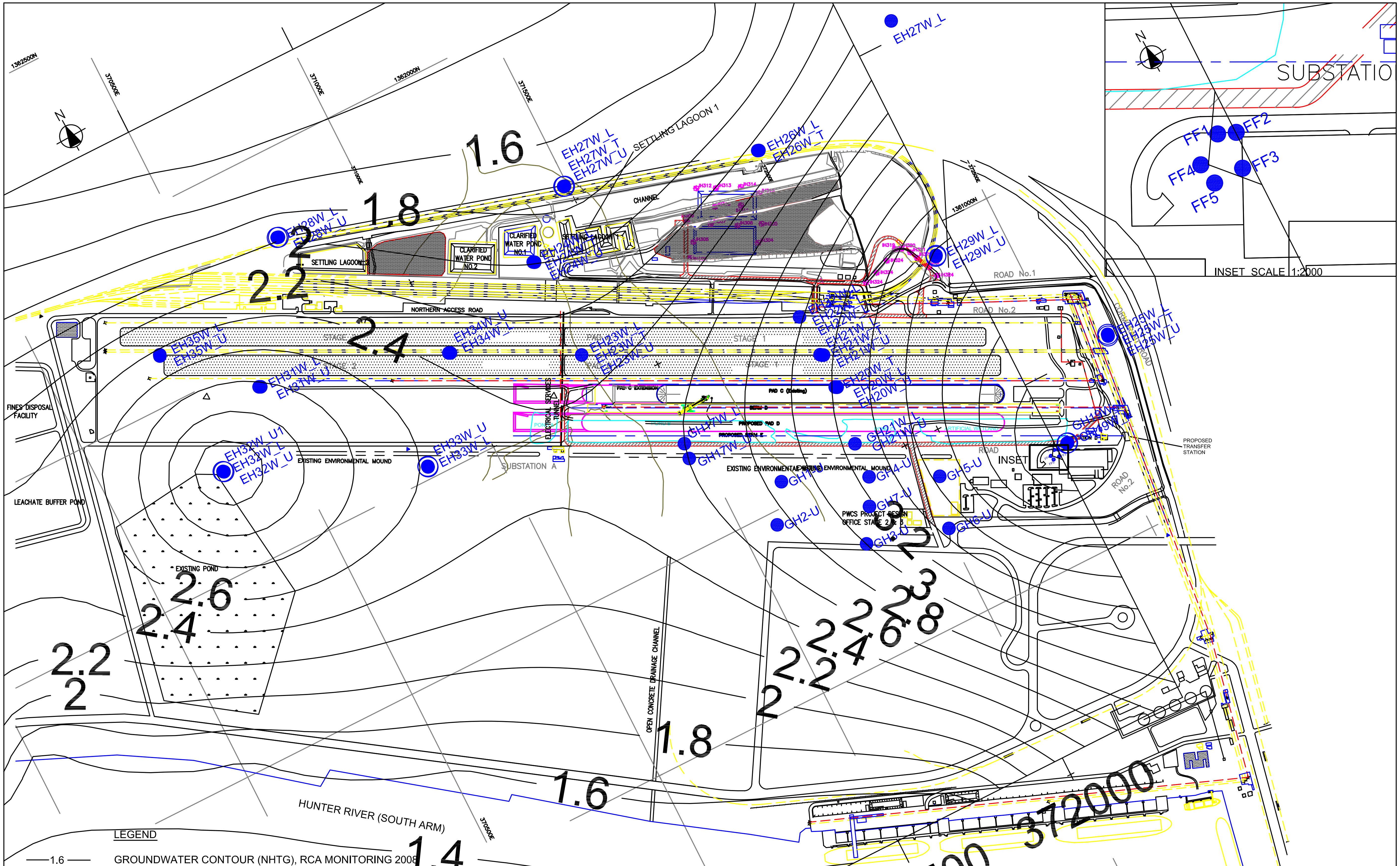
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some later stage, well after the event.

Reproduction of Information for Contractual Purposes

Attention is drawn to the document "Guidelines for the Provision of Geotechnical Information in Tender Documents", published by the Institution of Engineers, Australia. Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. The Company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Site Inspection

The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.



LEGEND

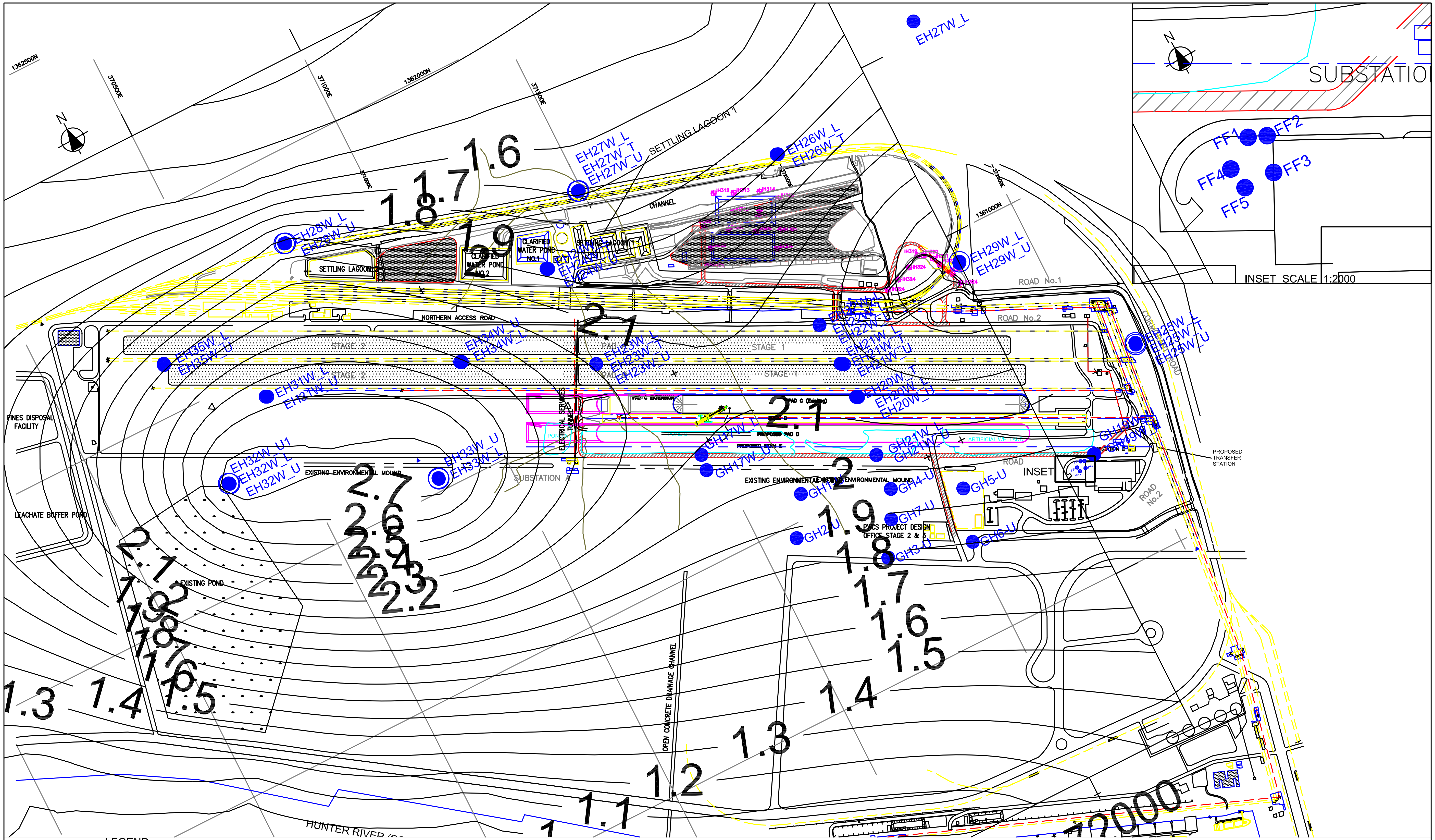
—1.6— GROUNDWATER CONTOUR (NHTG), RCA MONITORING 2008

APPROXIMATE LOCATION OF GROUNDWATER MONITORING WELL WITH WATER LEVEL MEASUREMENT

APPROXIMATE LOCATION OF GROUNDWATER MONITORING WELL

DRAWING ADAPTED FROM PLAN BY PORT WARATAH COAL SERVICES LIMITED REF KL41580 REV 2 AND PLANS SUPPLIED BY CLIENT, REFS KL 43401-02 AND KL43042-02

Douglas Partners Geotechnics Environment Groundwater	*Sydney, Newcastle, Brisbane, Melbourne, Perth, Wyong, Campbelltown, Townsville, Cairns, Wollongong, Darwin*	TITLE: GROUNDWATER CONTOURS - UPPER FILL AT KCT POTENTIAL GROUNDWATER IMPACTS, STAGE 4 PROJECT KOORAGANG COAL TERMINAL (KCT), KOORAGANG ISLAND	CLIENT: UMWELT; REF:P:\49425\DRAWINGS\49425-1; DRAWN BY: PLH; SCALE: 1:10000; PROJECT No: 49425; OFFICE: NEWCASTLE; APPROVED BY: ; DATE: ; DRAWING No: 1



LEGEND

1.6 — GROUNDWATER CONTOUR (NHTG), RCA MONITORING 2008

APPROXIMATE LOCATION OF GROUNDWATER MONITORING WELL WITH WATER LEVEL MEASUREMENTS

APPROXIMATE LOCATION OF GROUNDWATER MONITORING WELL

DRAWING ADAPTED FROM PLAN BY PORT WARATAH COAL SERVICES LIMITED REF KL41580 REV 2 AND PLANS SUPPLIED BY CLIENT, REFS KL 43401-02 AND KL43042-02

Douglas Partners Geotechnics Environment Groundwater	Sydney, Newcastle, Brisbane, Melbourne, Perth, Wyong, Campbelltown, Townsville, Cairns, Wollongong, Darwin	TITLE: GROUNDWATER CONTOURS - ESTUARINE AQUIFER AT KCT POTENTIAL GROUNDWATER IMPACTS, STAGE 4 PROJECT KOORAGANG COAL TERMINAL (KCT), KOORAGANG ISLAND	CLIENT: UMWELT

CLIENT: UMWELT			REF:P:\49425\DRAWINGS\49425-2
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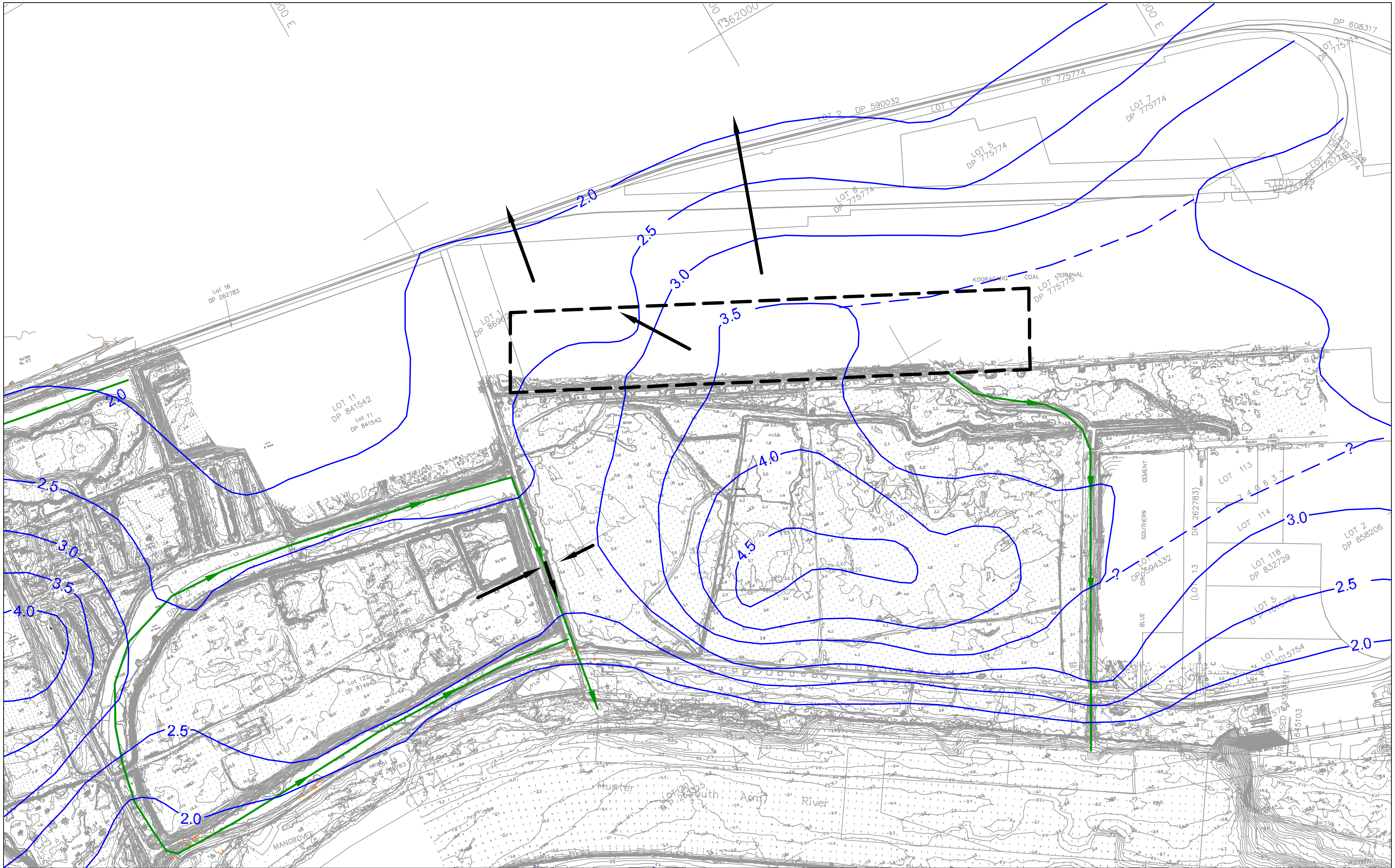
APPROVED BY:		DATE:	DRAWING No: 2



LEGEND

GROUNDWATER FLOW DIRECTION

SURFACE DRAIN SHOWING FLOW DIRECTION

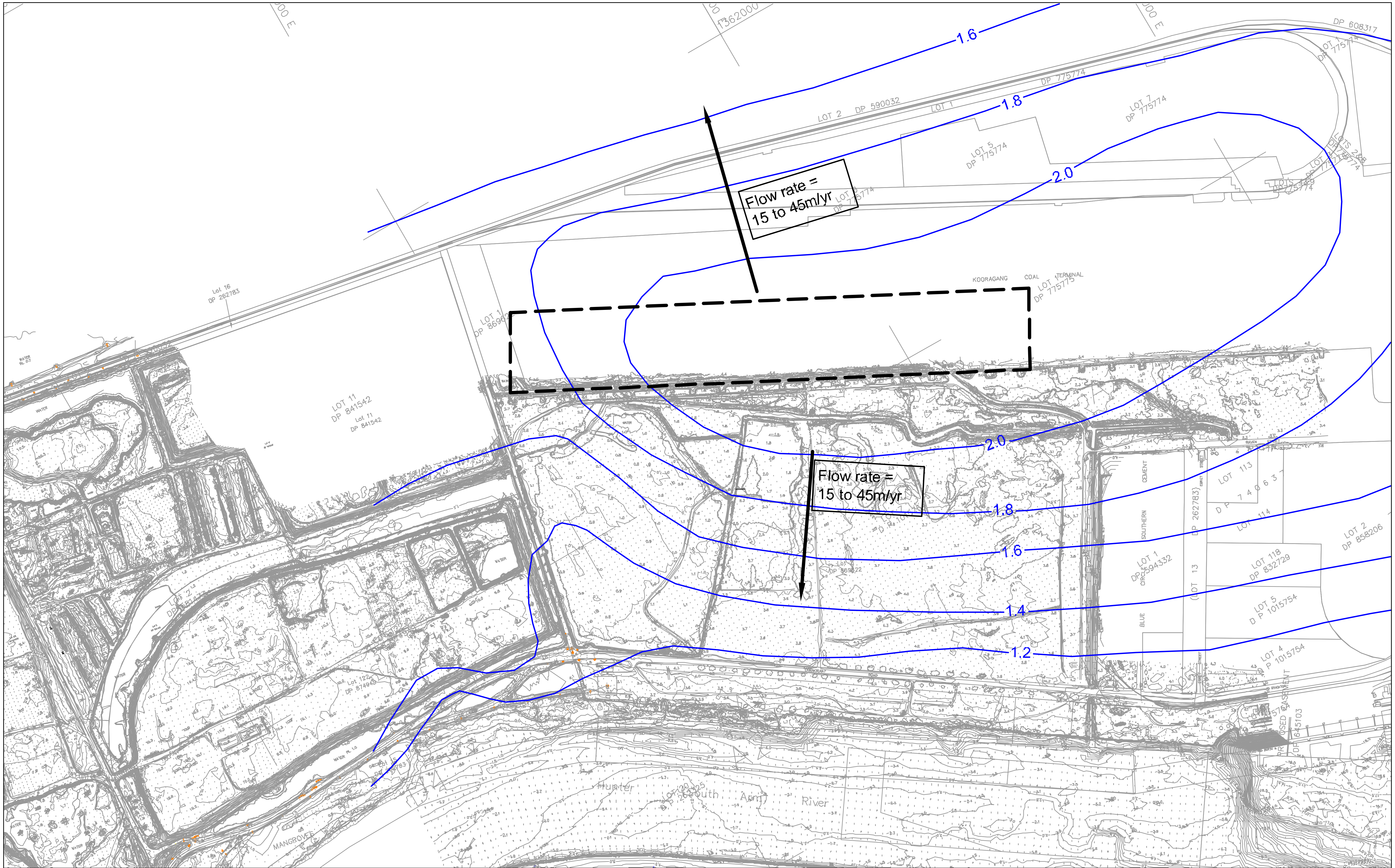
SITE AREA

2.5 GROUNDWATER CONTOUR (AHD)

0 100 200 300 400 500 600 700 800 900 1000 m

SCALE 1:10000 (A3 SHEET)

Douglas Partners Geotechnics Environment Groundwater	Sydney, Newcastle, Brisbane, Melbourne, Perth, Wyong, Campbelltown, Townsville, Cairns, Wollongong, Darwin	TITLE: GROUNDWATER CONTOURS IN FILL AQUIFER KOORAGANG COAL TERMINAL PADS C & D RAVEN STREET, KOORAGANG ISLAND	CLIENT: PORT WARATAH COAL SERVICES
			DRAWN BY: PLH	SCALE: 1:10000	PROJECT No: 39500A	OFFICE: NEWCASTLE
			APPROVED BY:	DATE:	DRAWING No: 13



LEGEND

GROUNDWATER FLOW DIRECTION

Flow rate = 15 to 45m/yr — GROUNDWATER FLOW VELOCITY (m/yr)

SITE AREA

1.2 — GROUNDWATER CONTOUR (AHD)

0 100 200 300 400 500 600 700 800 900 1000 m

SCALE 1:10000 (A3 SHEET)

Douglas Partners
Geotechnics Environment Groundwater

Sydney, Newcastle, Brisbane, Melbourne, Perth, Wyong, Campbelltown, Townsville, Cairns, Wollongong, Darwin

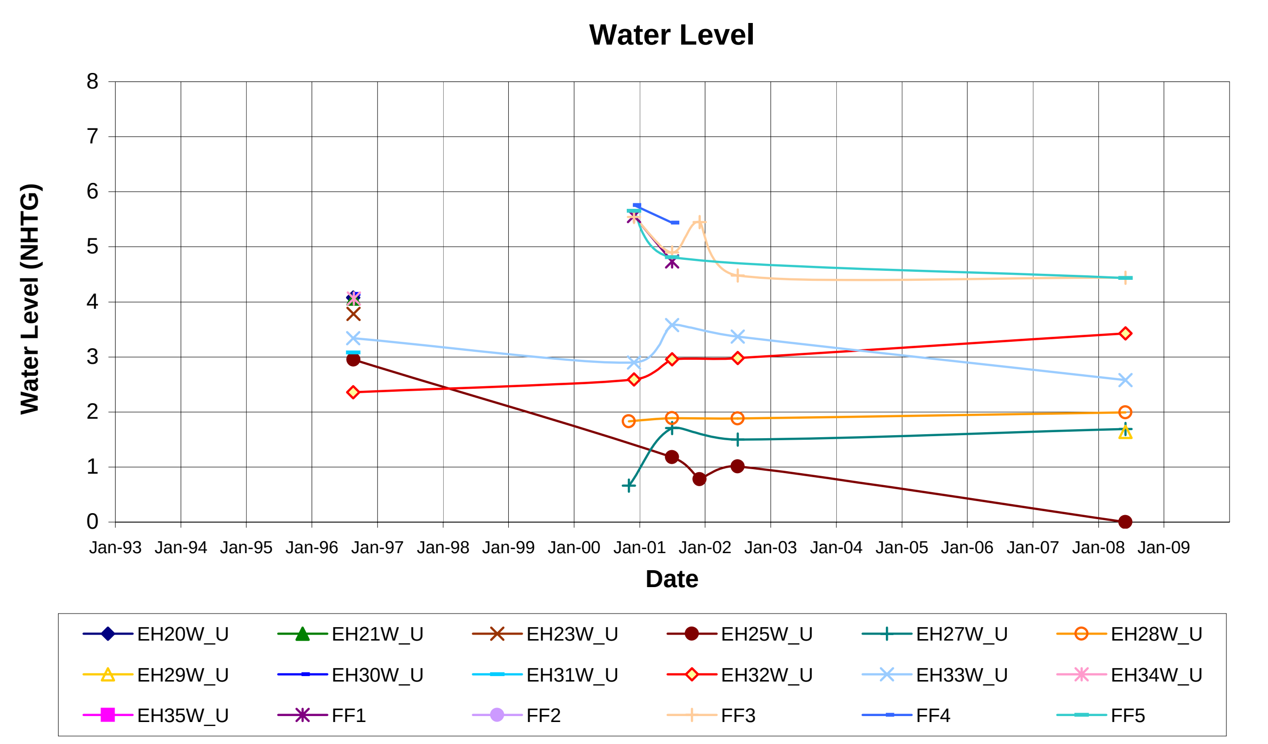
TITLE: GROUNDWATER CONTOURS IN ESTUARINE AQUIFER
KOORAGANG COAL TERMINAL PADS C & D
RAVEN STREET, KOORAGANG ISLAND

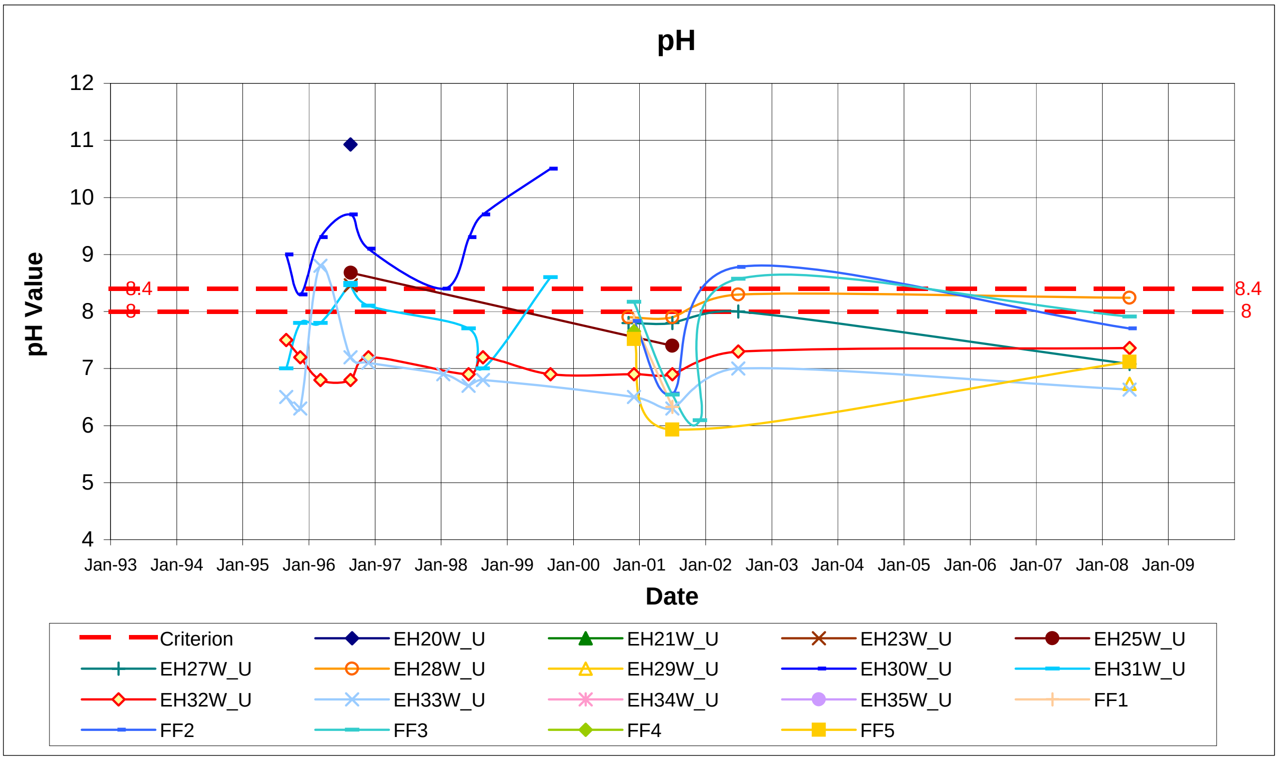
CLIENT: PORT WARATAH COAL SERVICES			
DRAWN BY: PLH	SCALE: 1:10000	PROJECT No: 39500A	OFFICE: NEWCASTLE
APPROVED BY:		DATE:	DRAWING No: 14

PWCS Kooragang Coal Terminal

GROUNDWATER QUALITY MONITORING

Aquifer: **Upper (Fill) Aquifer** **Project:** **49322**





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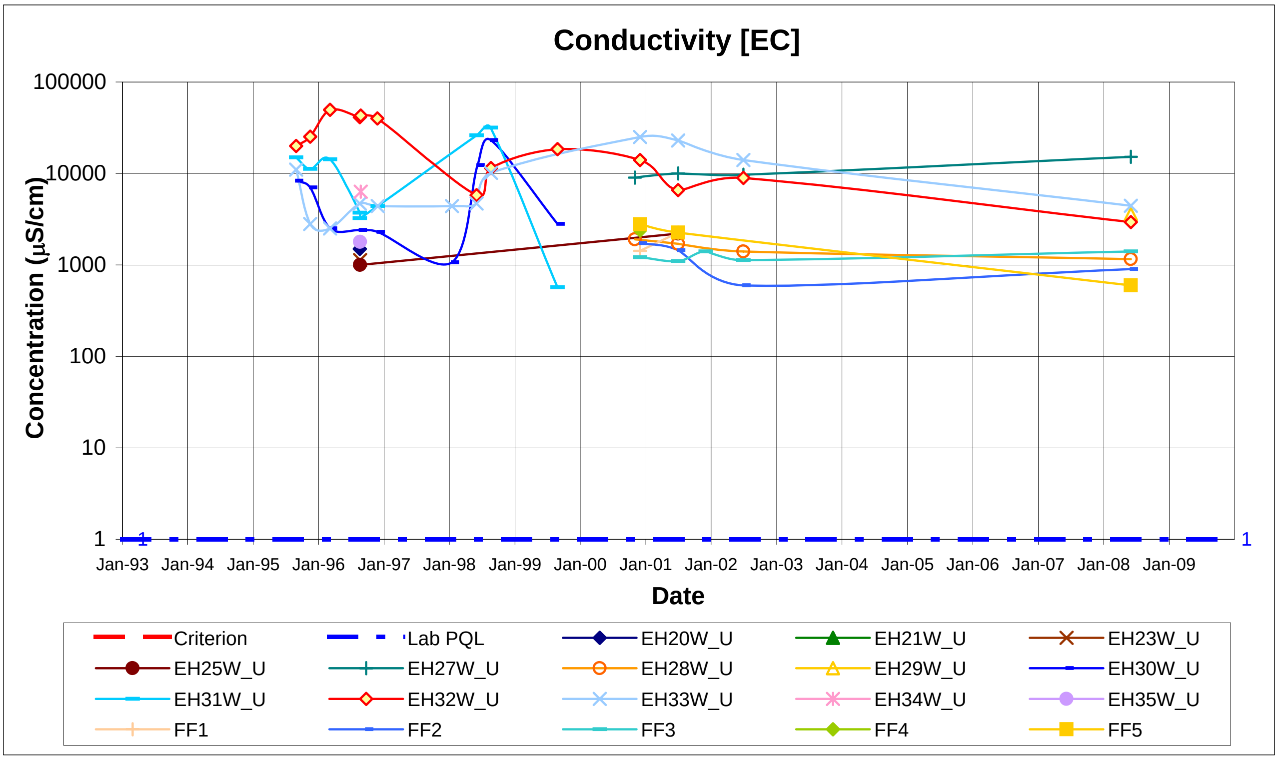
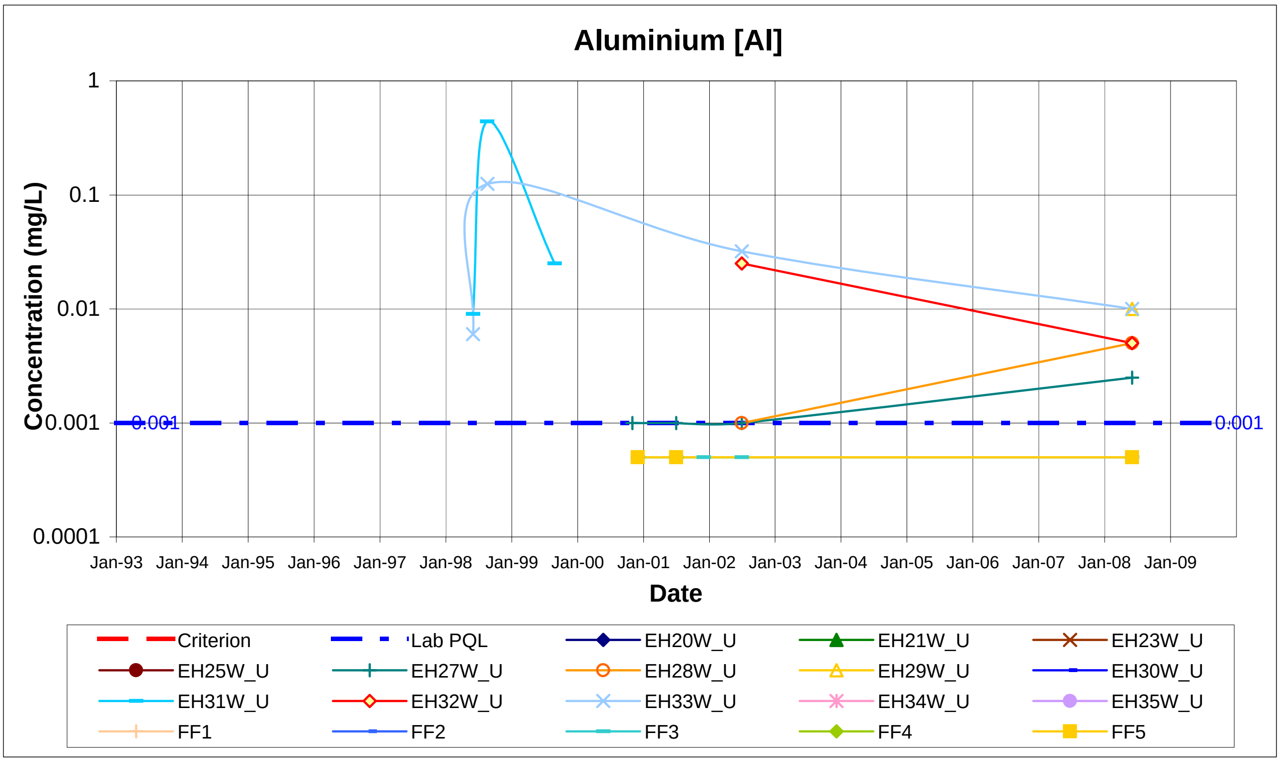
PWCS Kooragang Coal Terminal

GROUNDWATER QUALITY MONITORING

Aquifer: Upper (Fill) Aquifer **Project:** 49322





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GROUNDWATER QUALITY MONITORING

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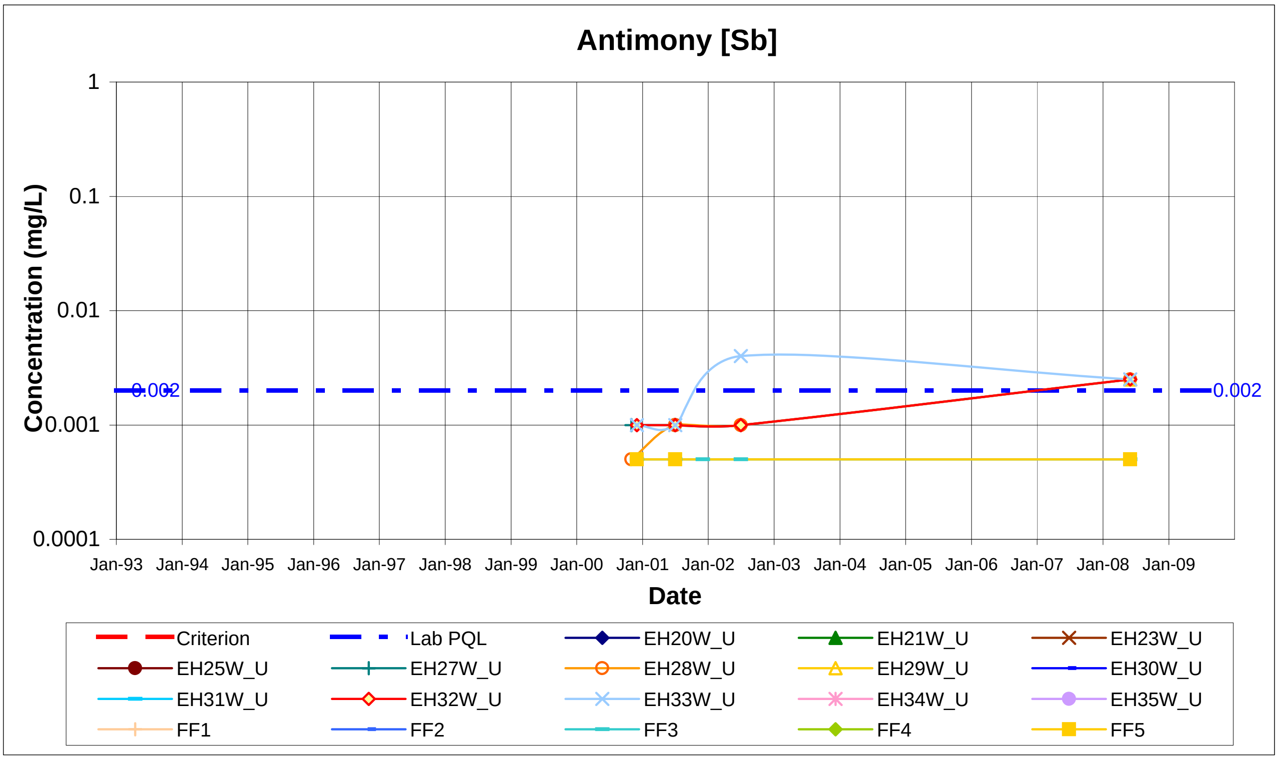

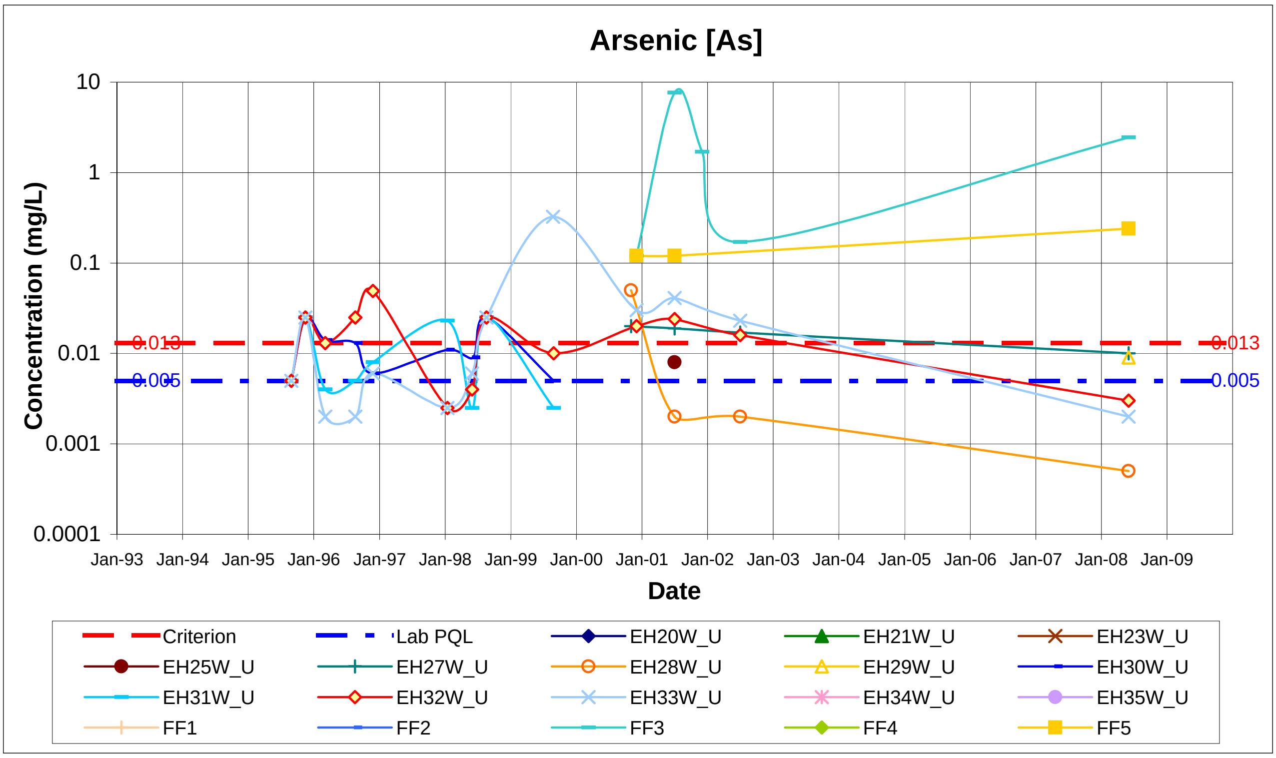

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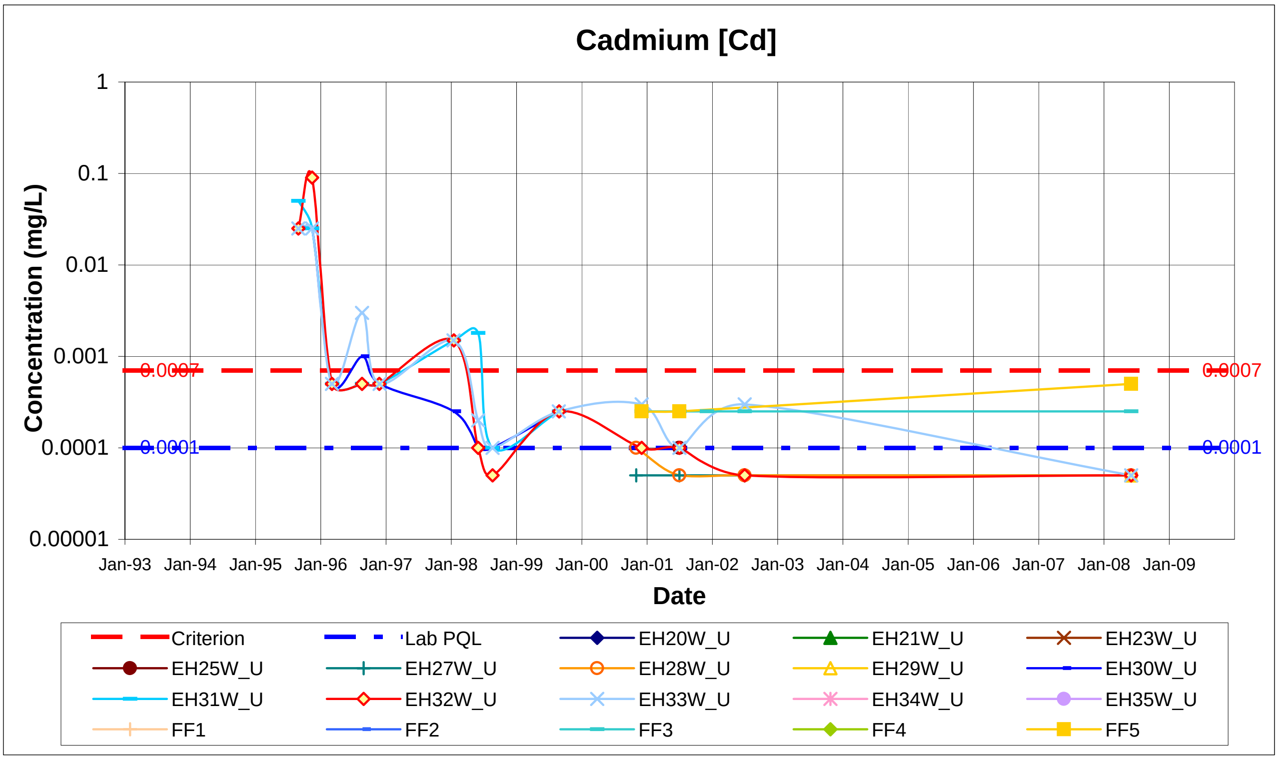

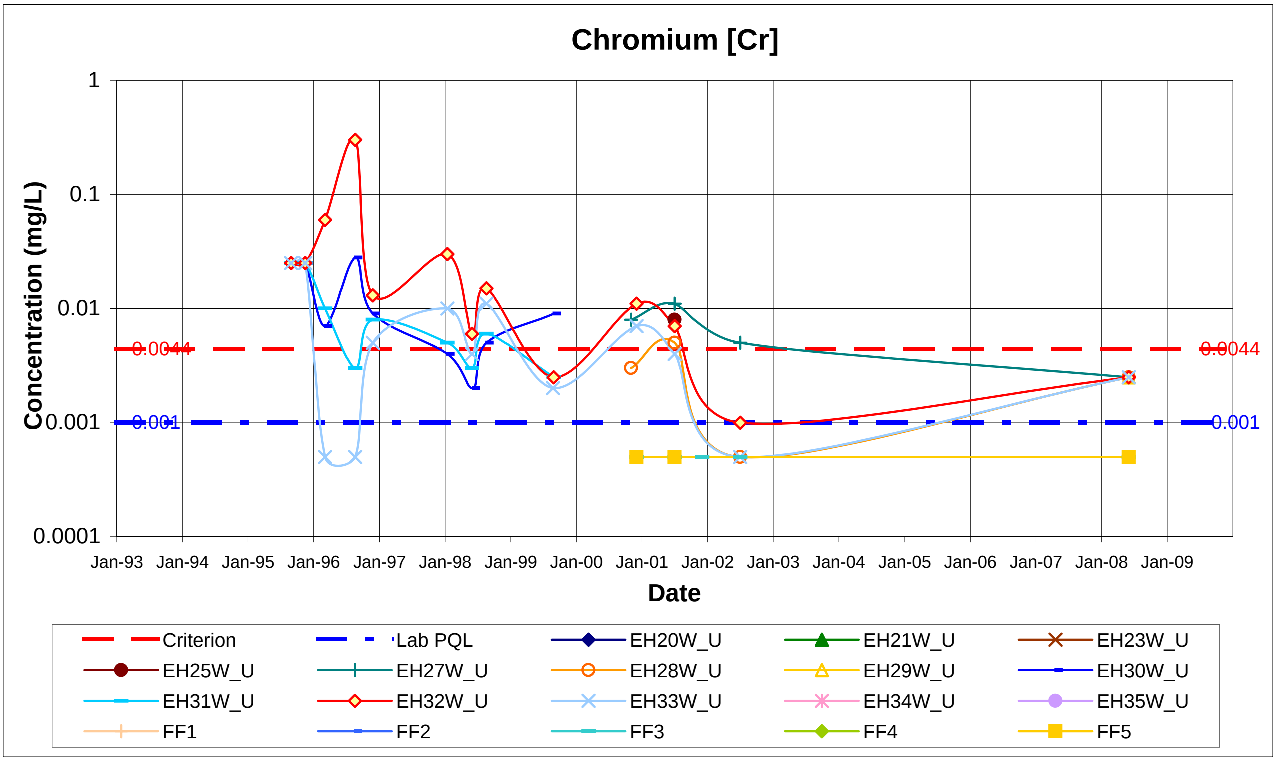

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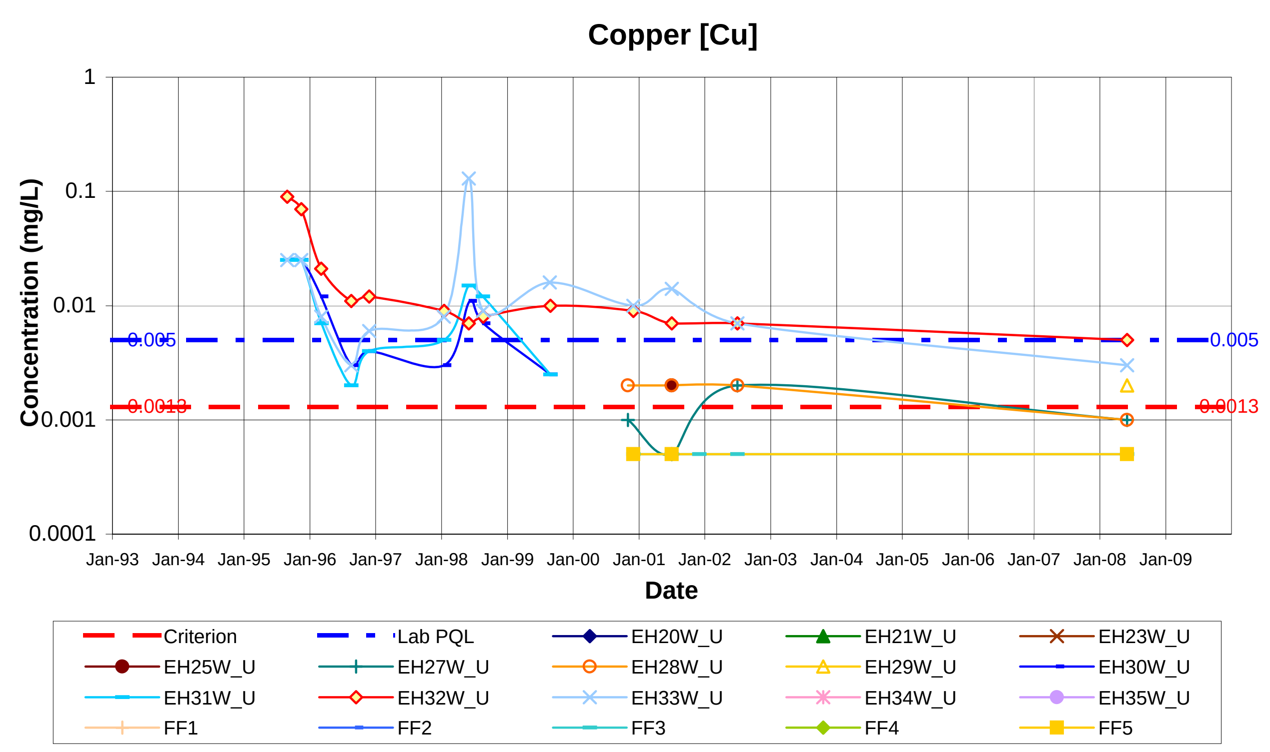

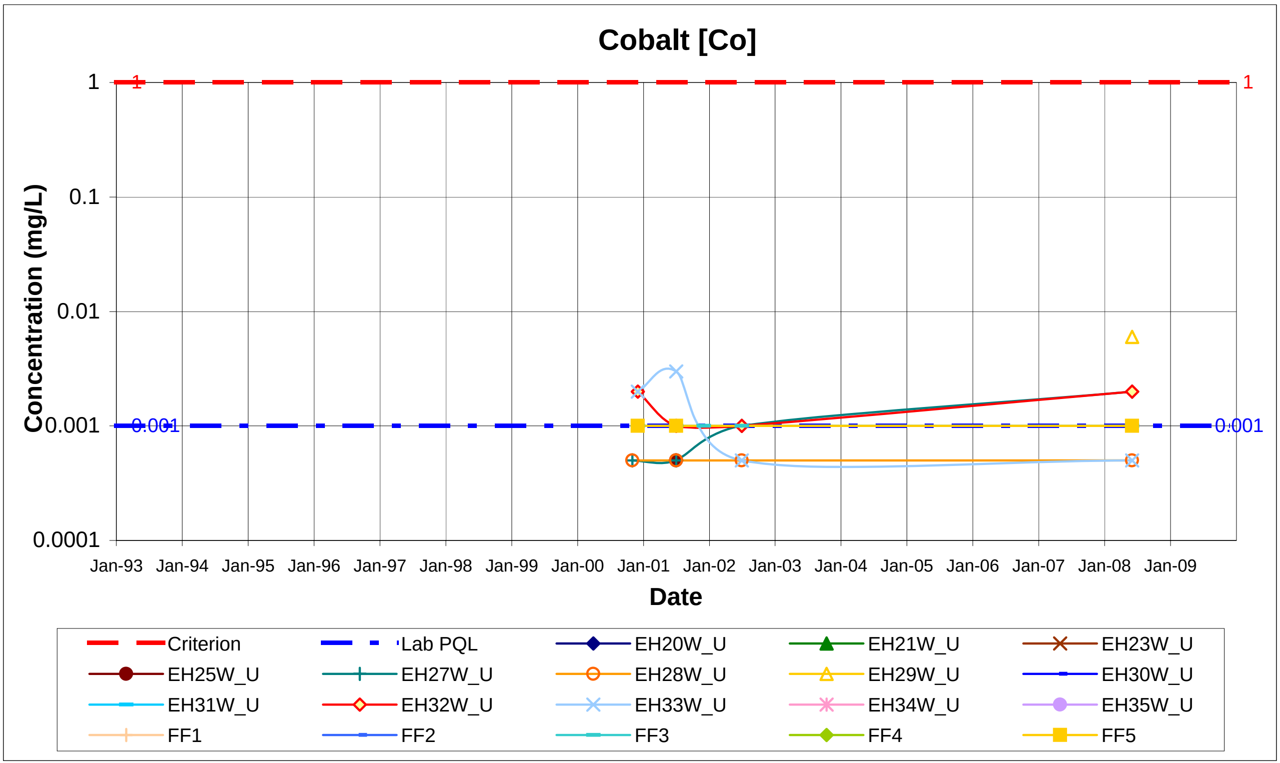

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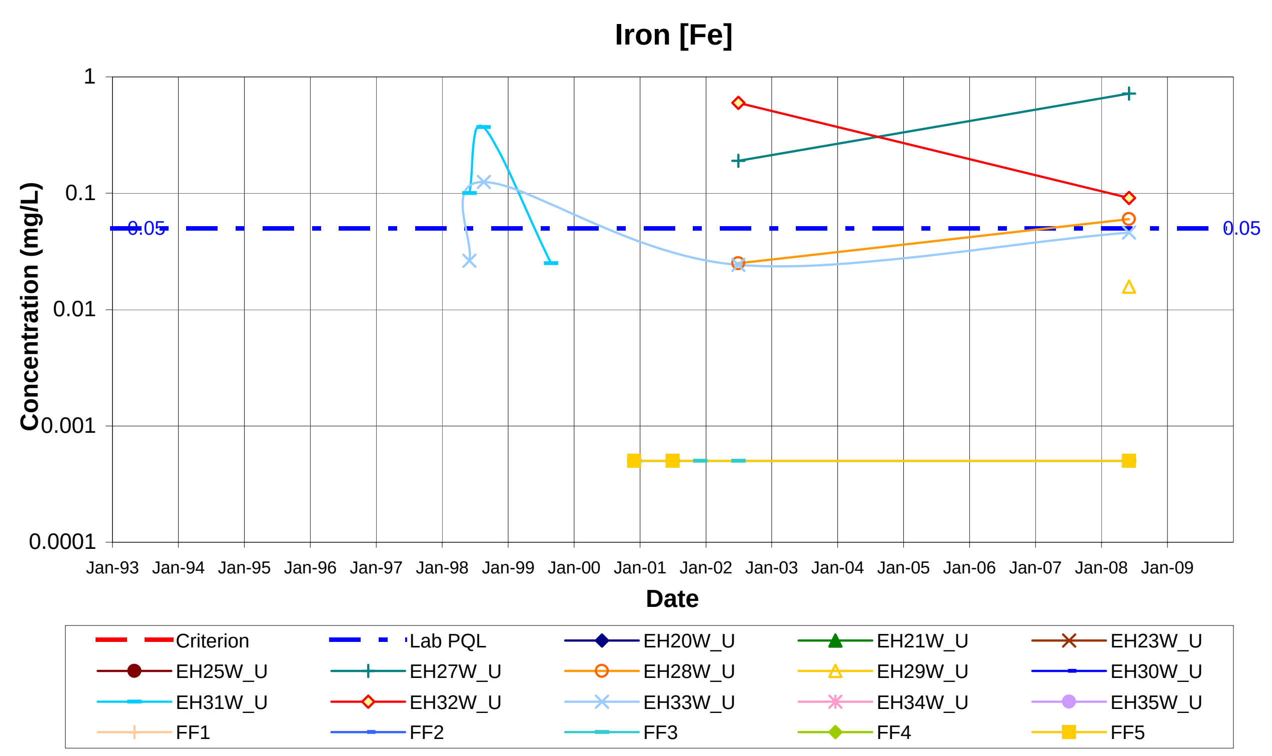

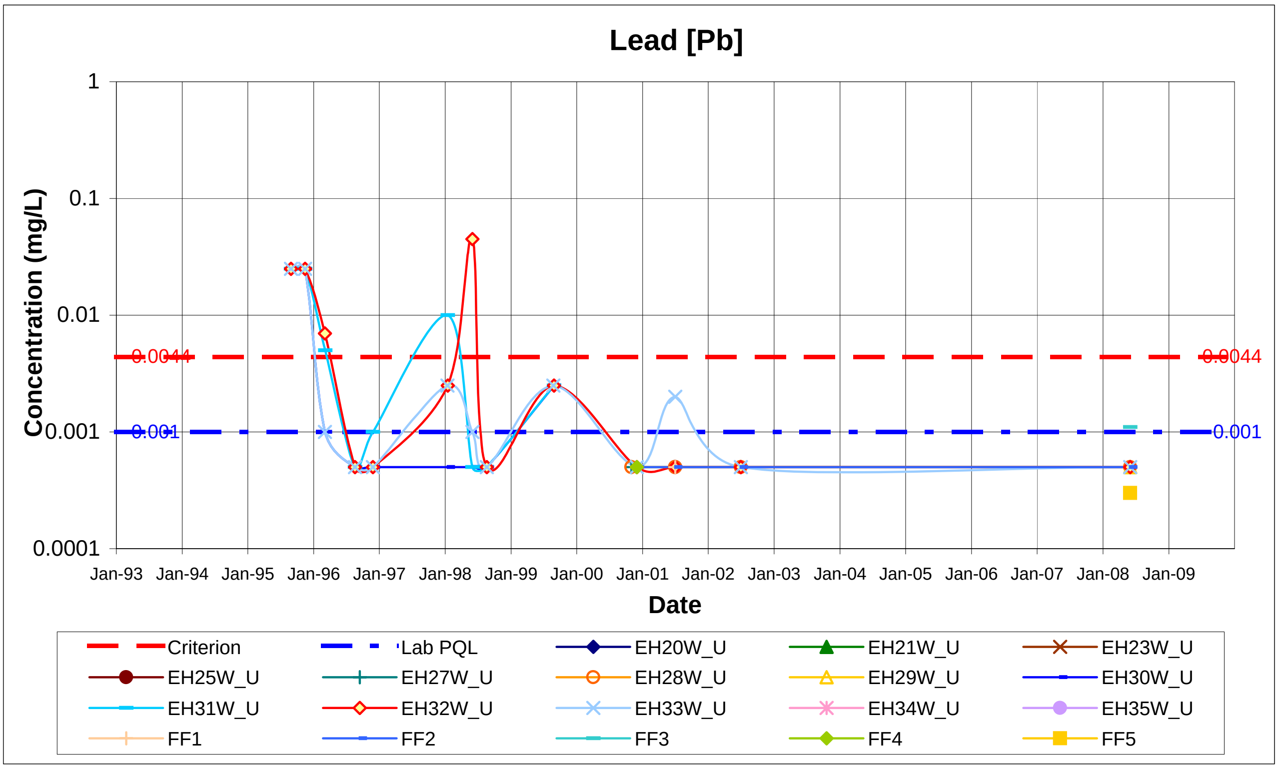

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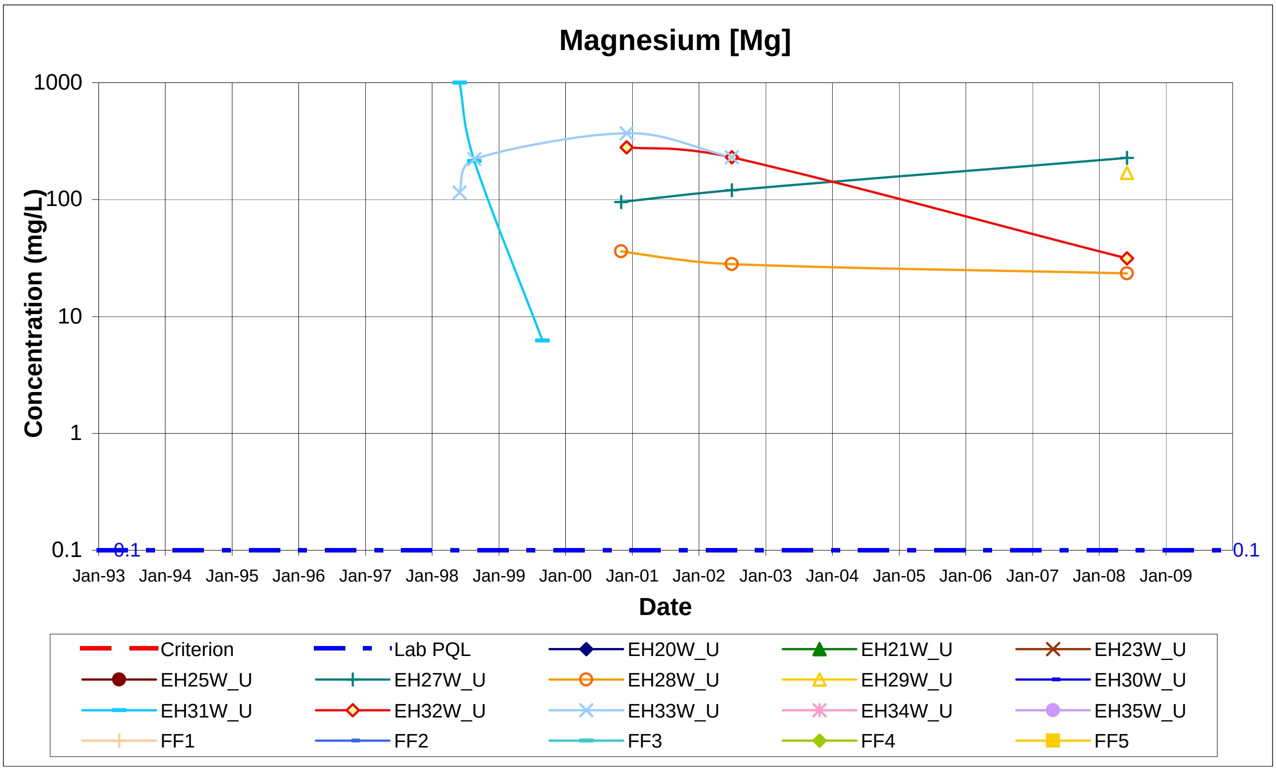

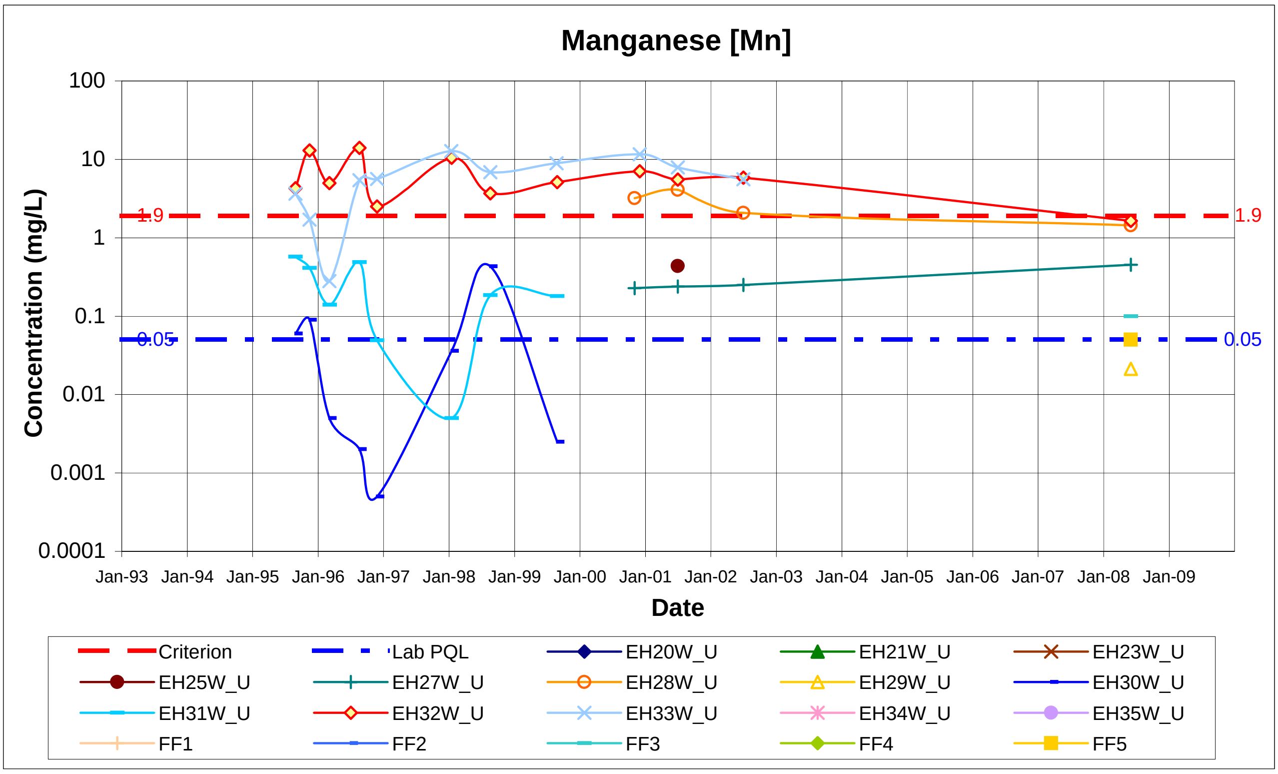

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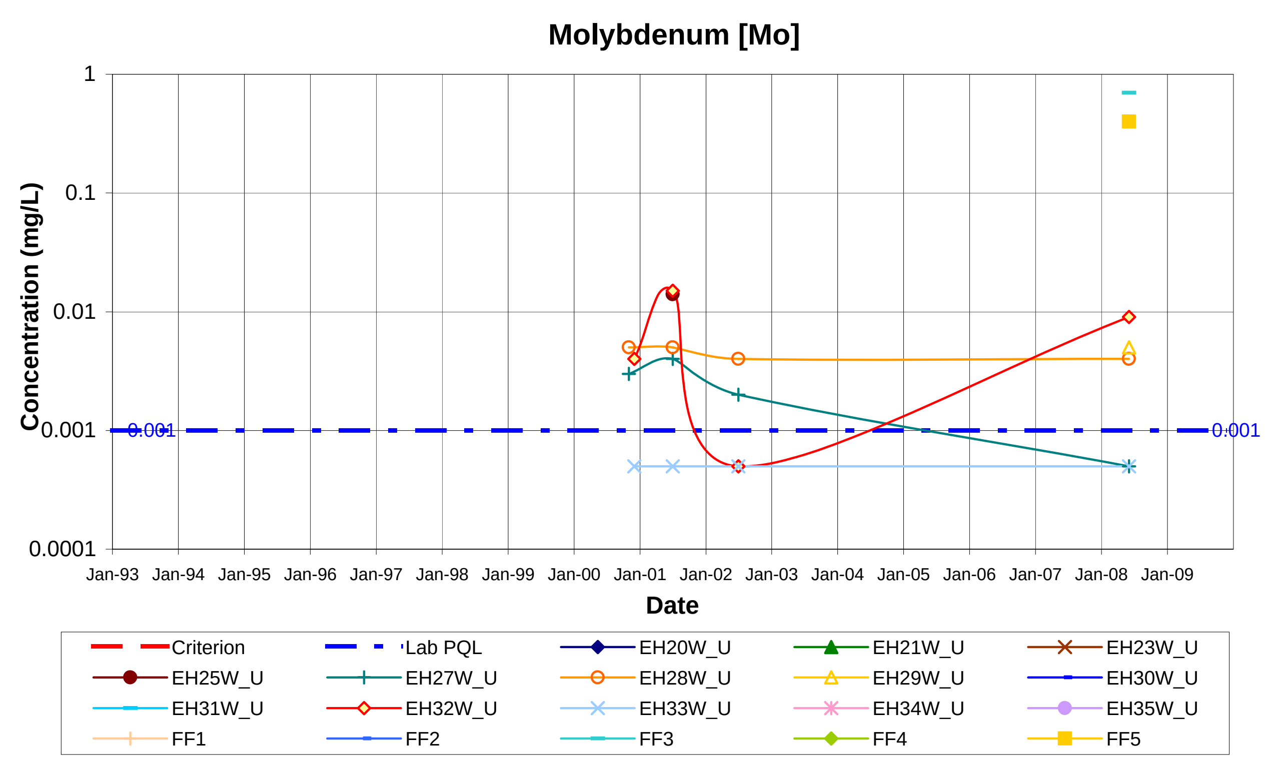

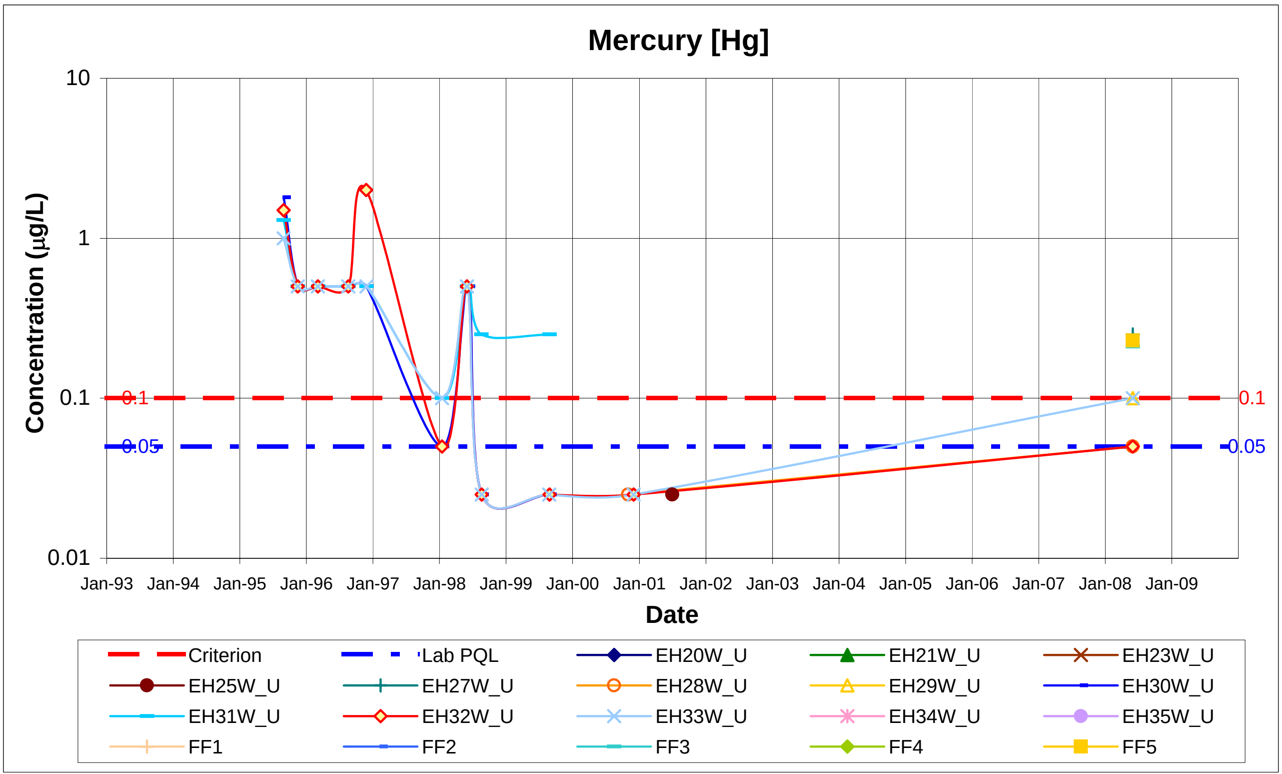

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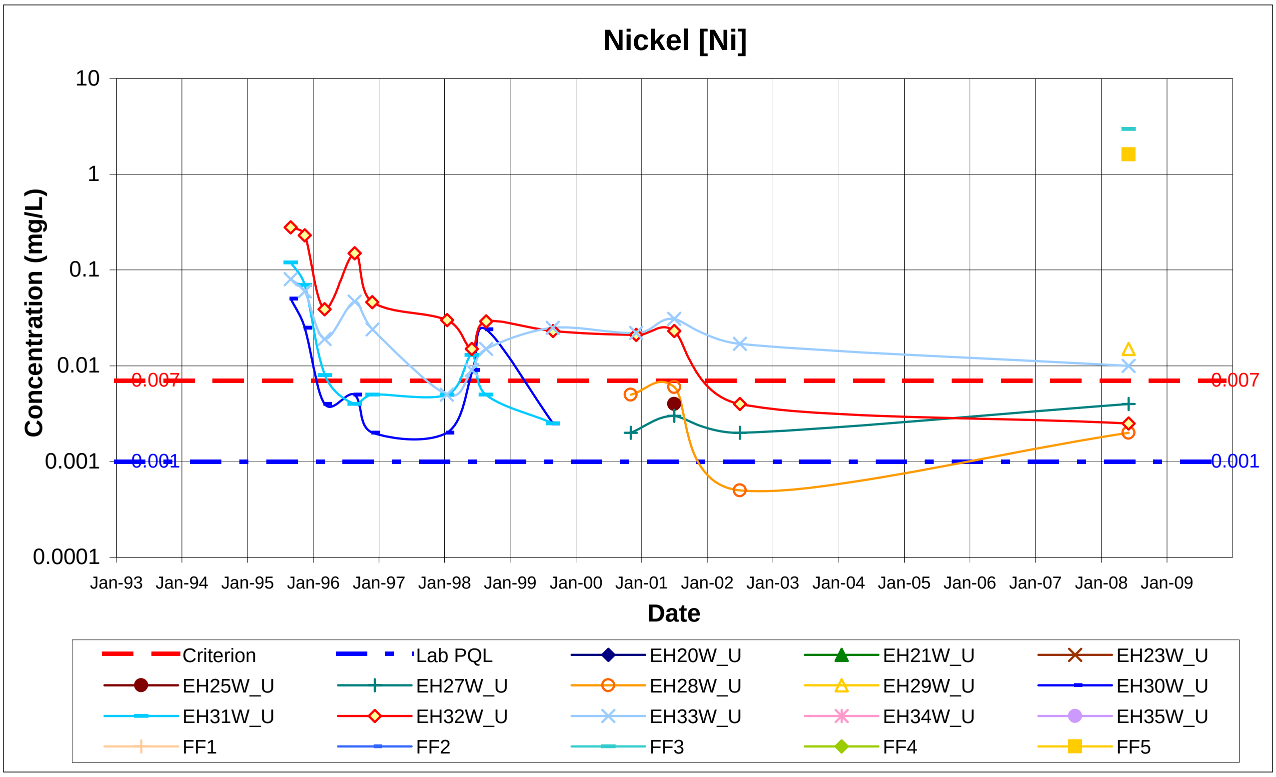

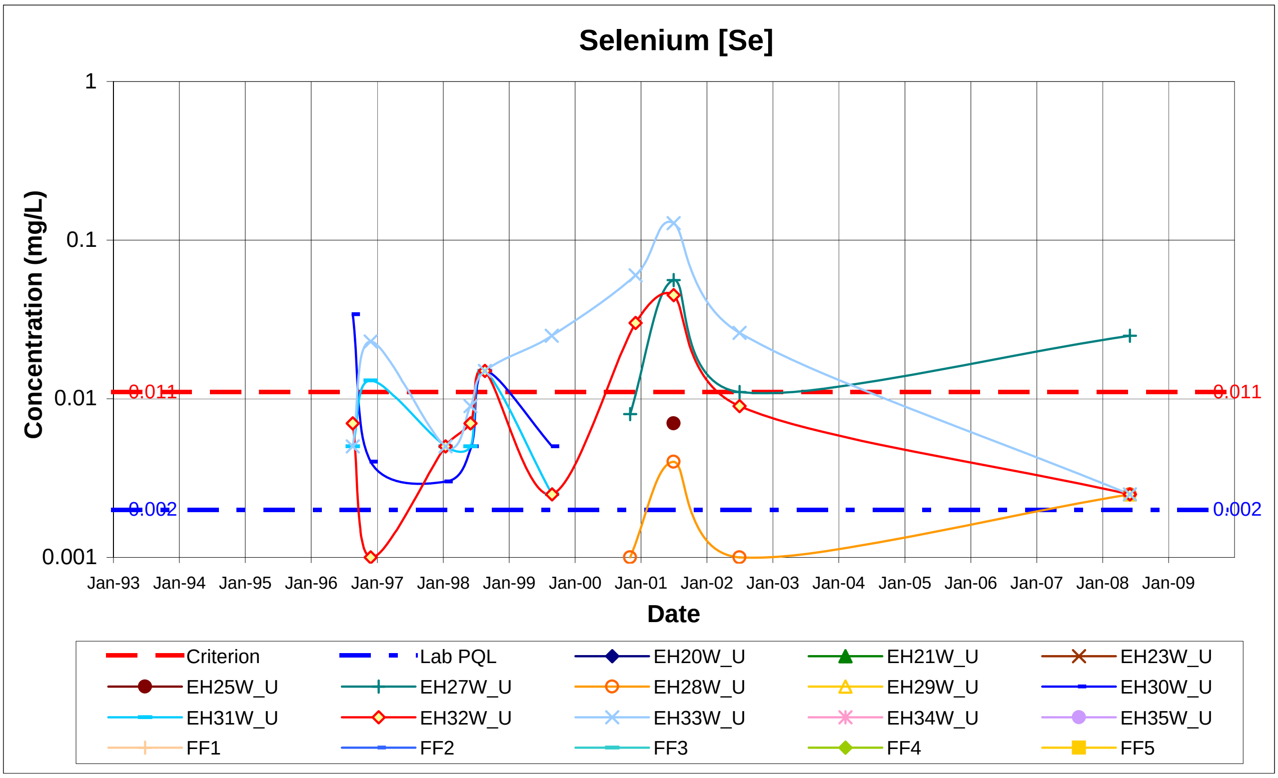

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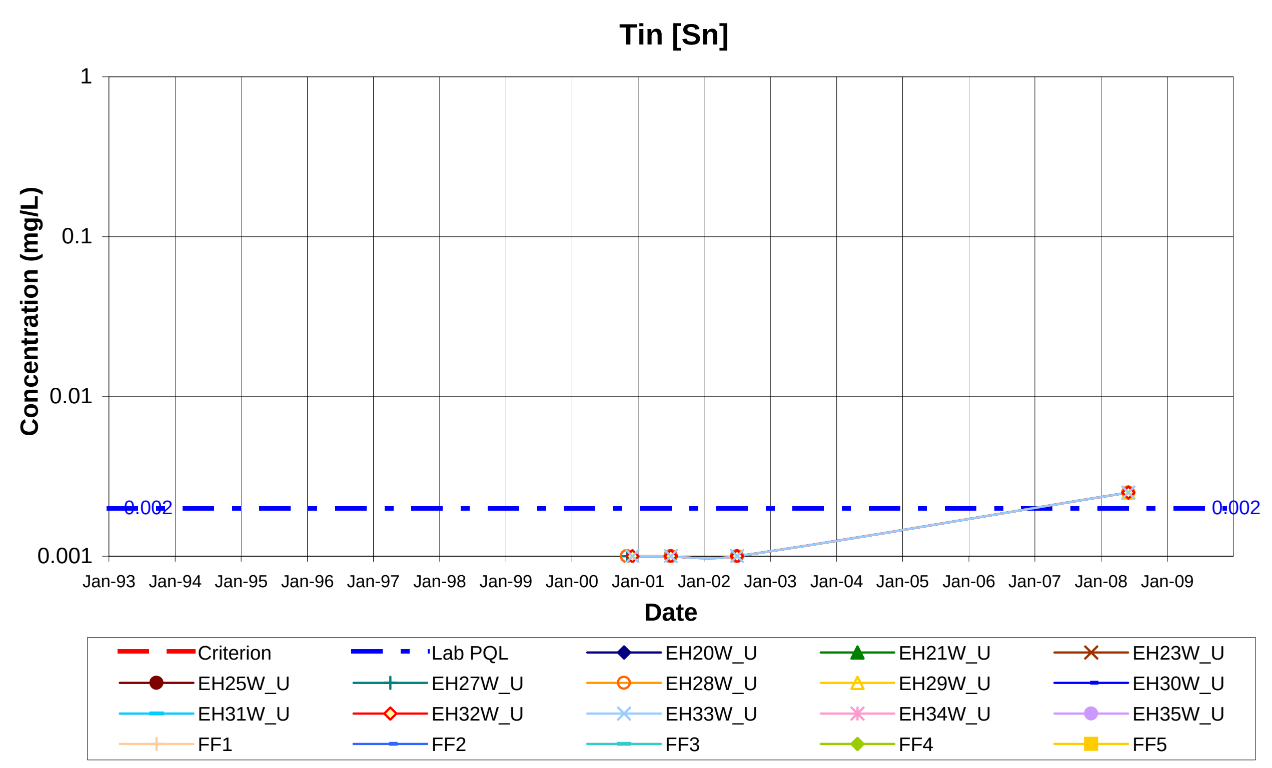

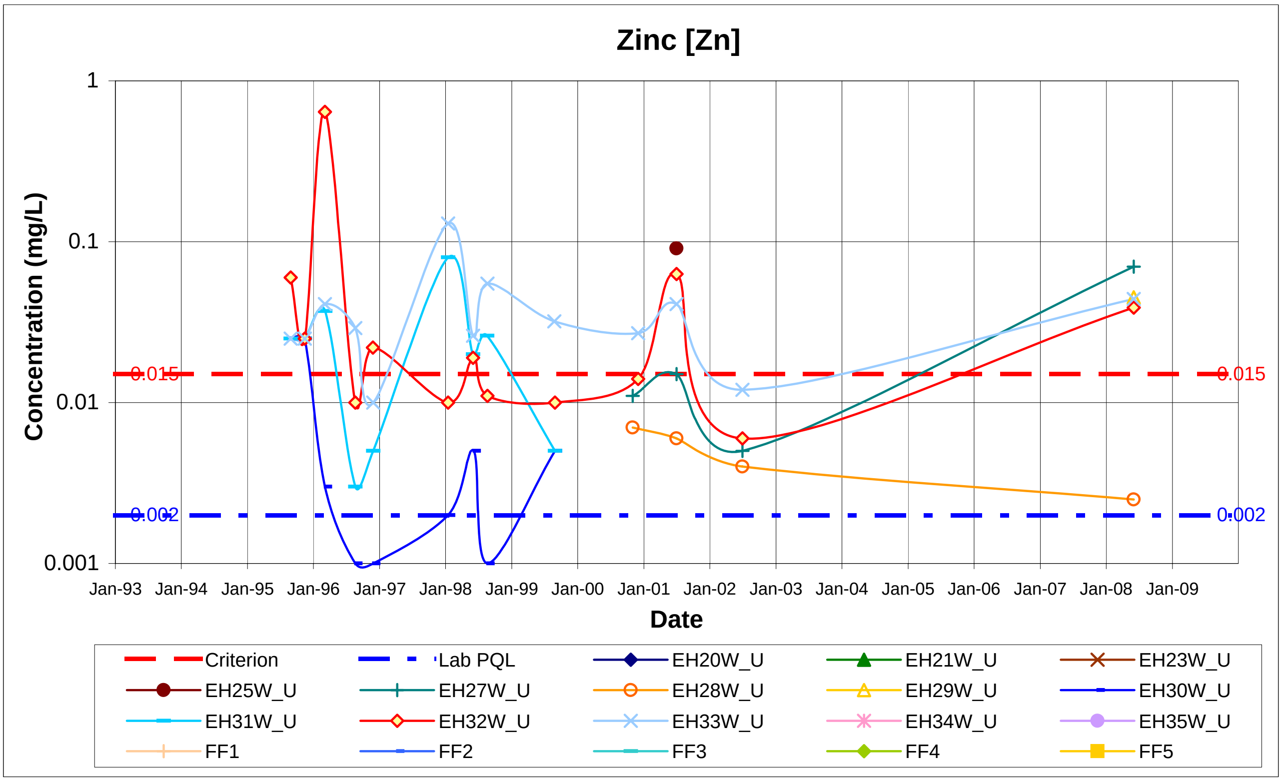

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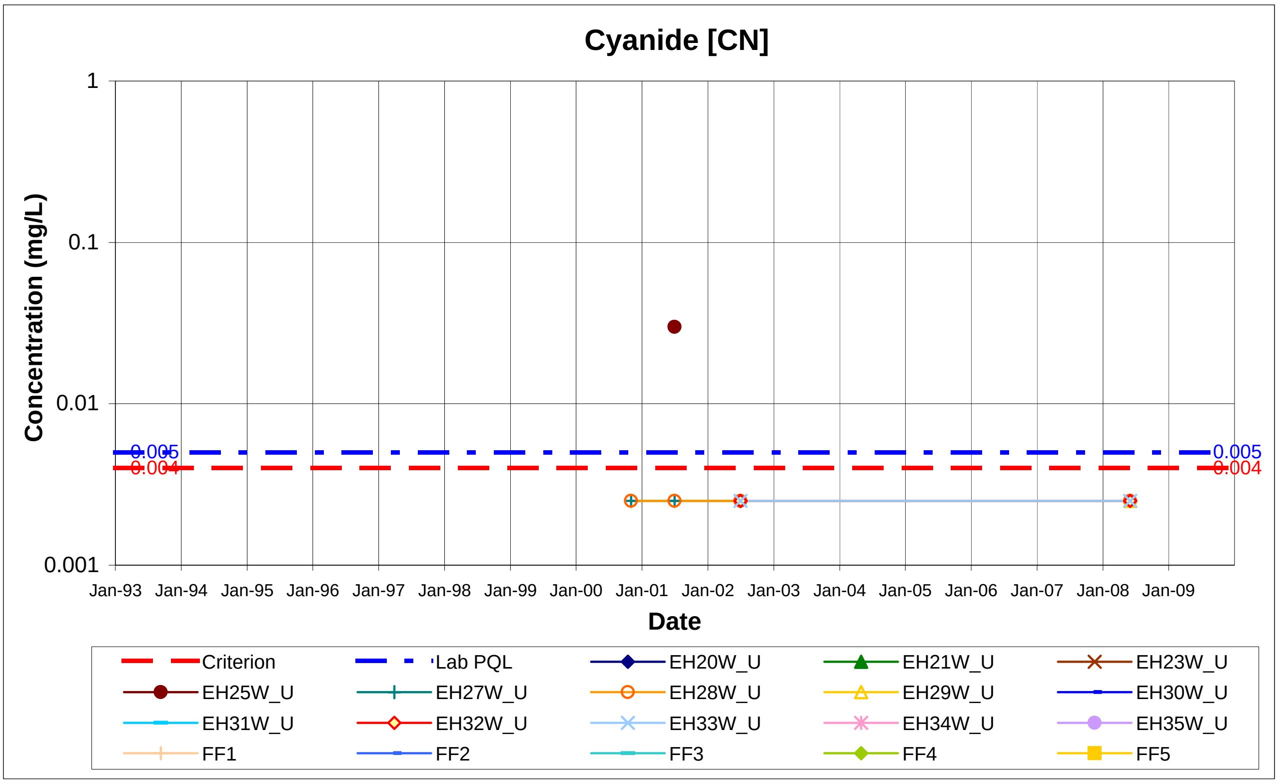

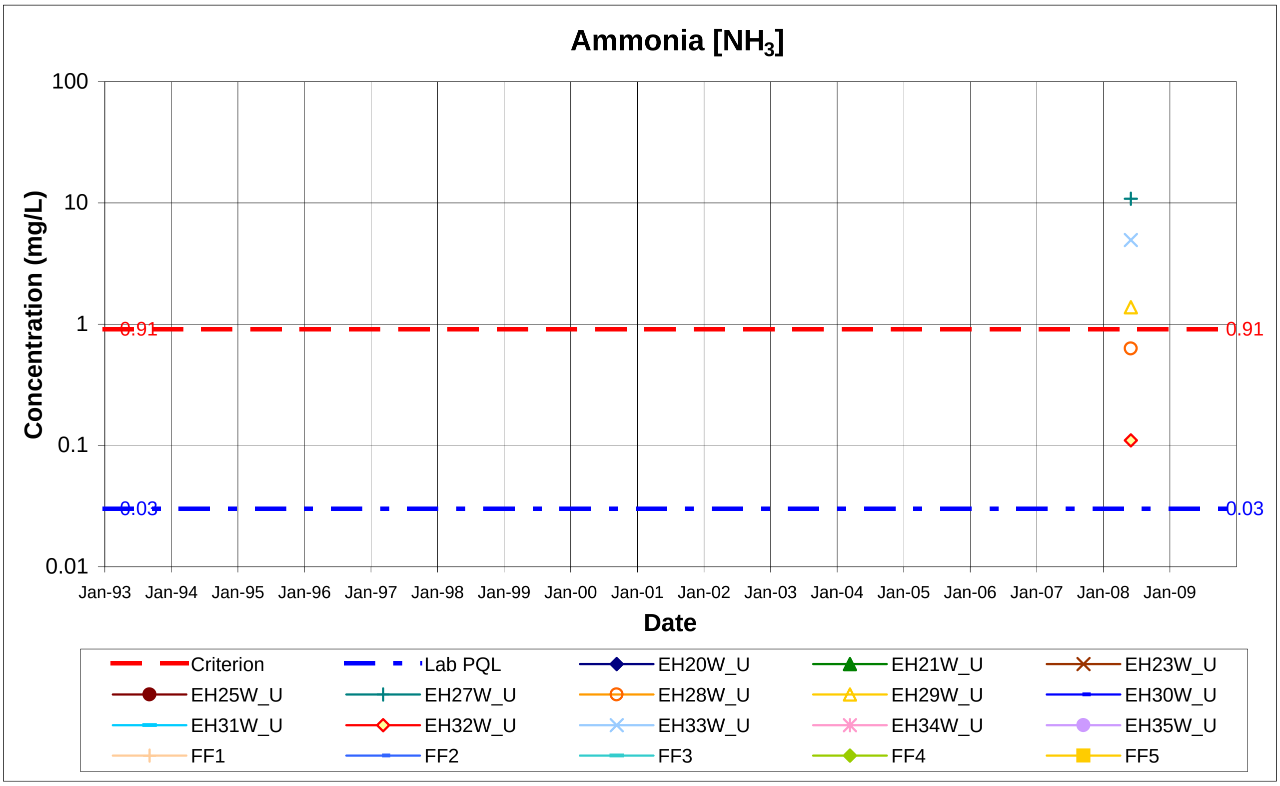

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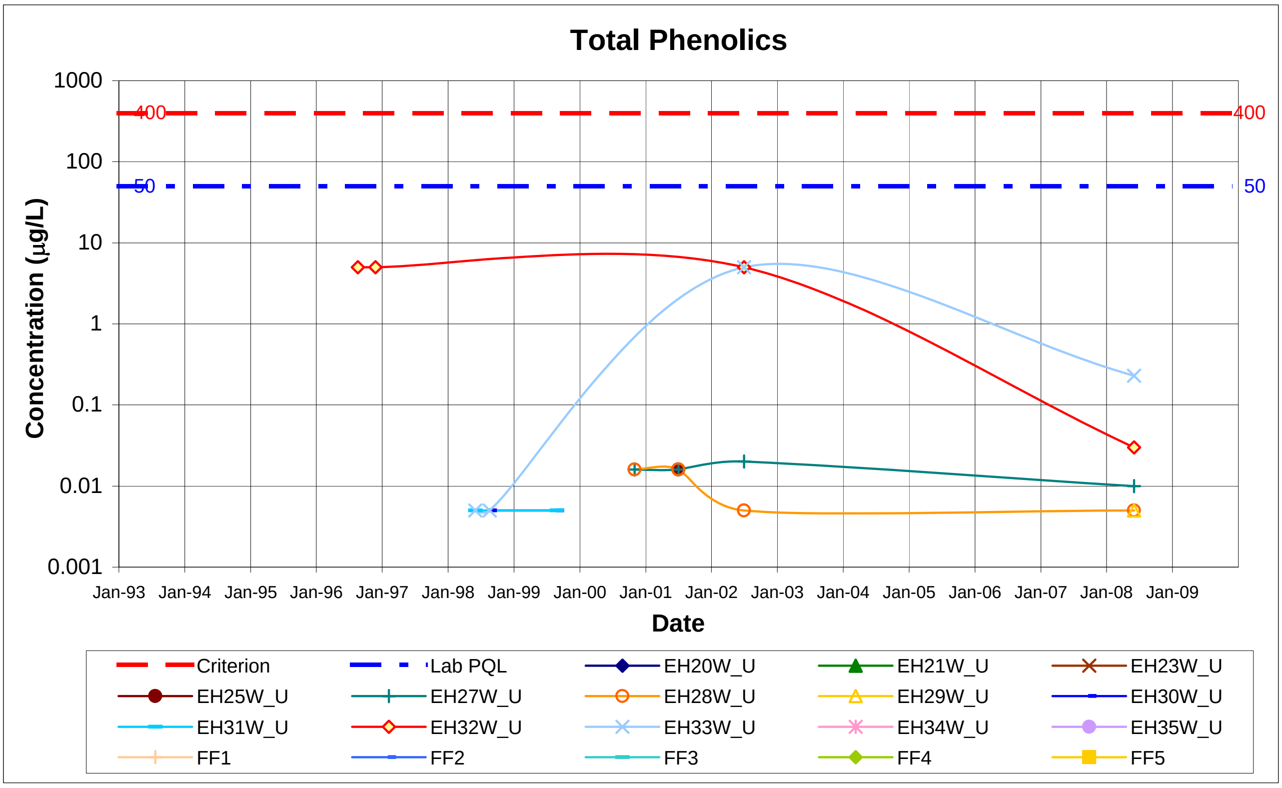

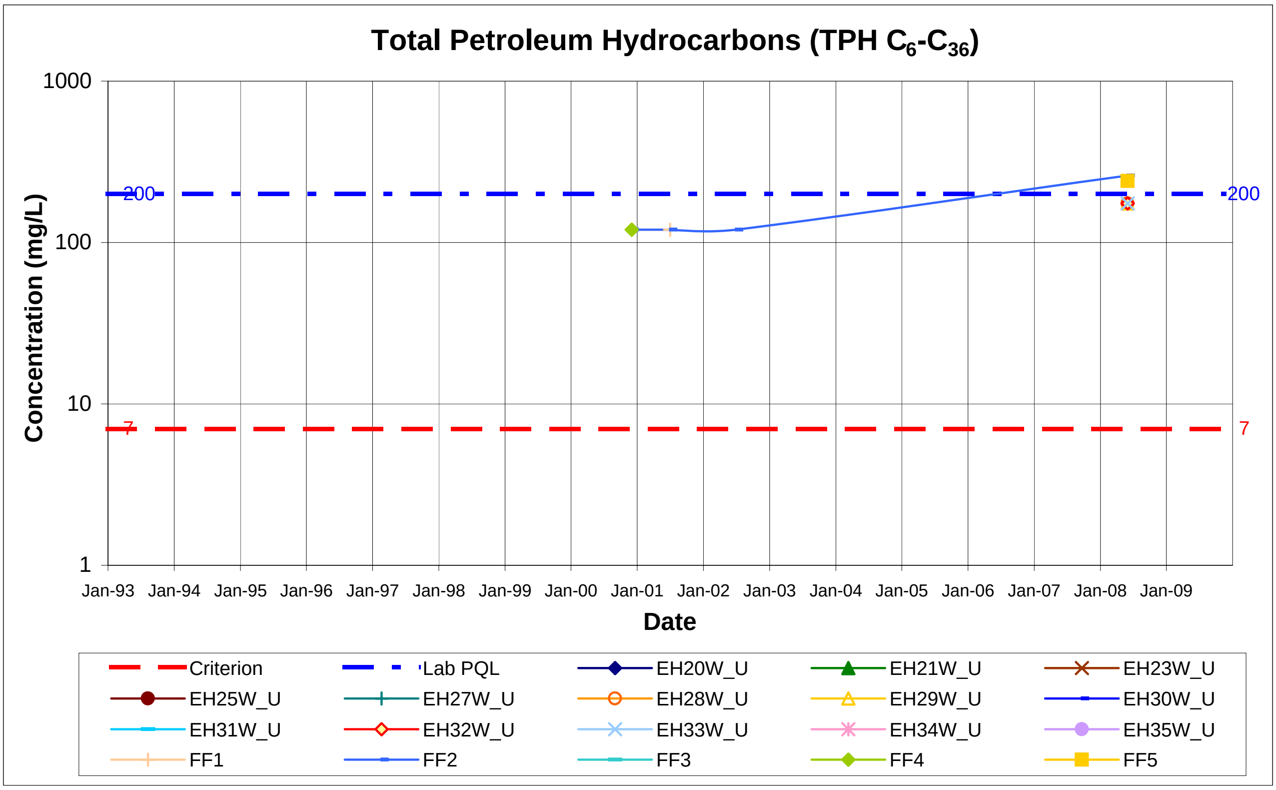

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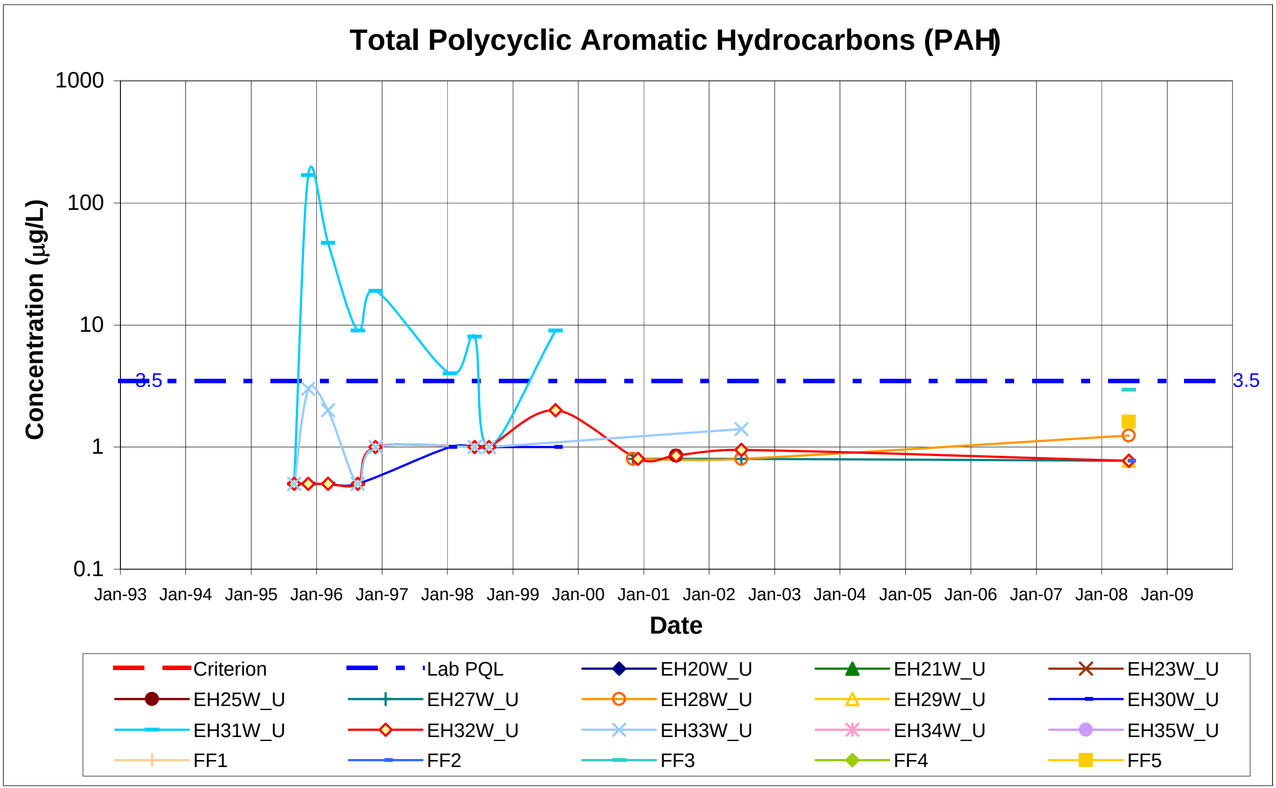

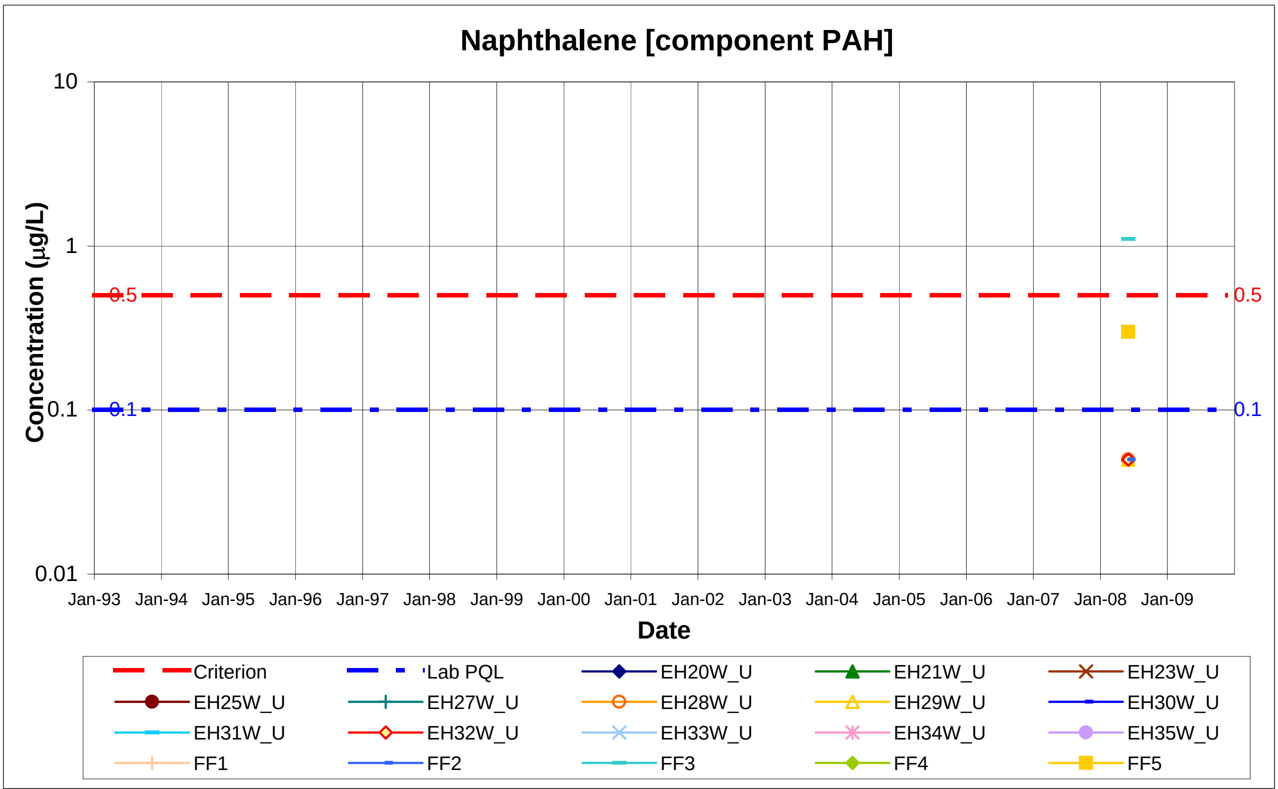

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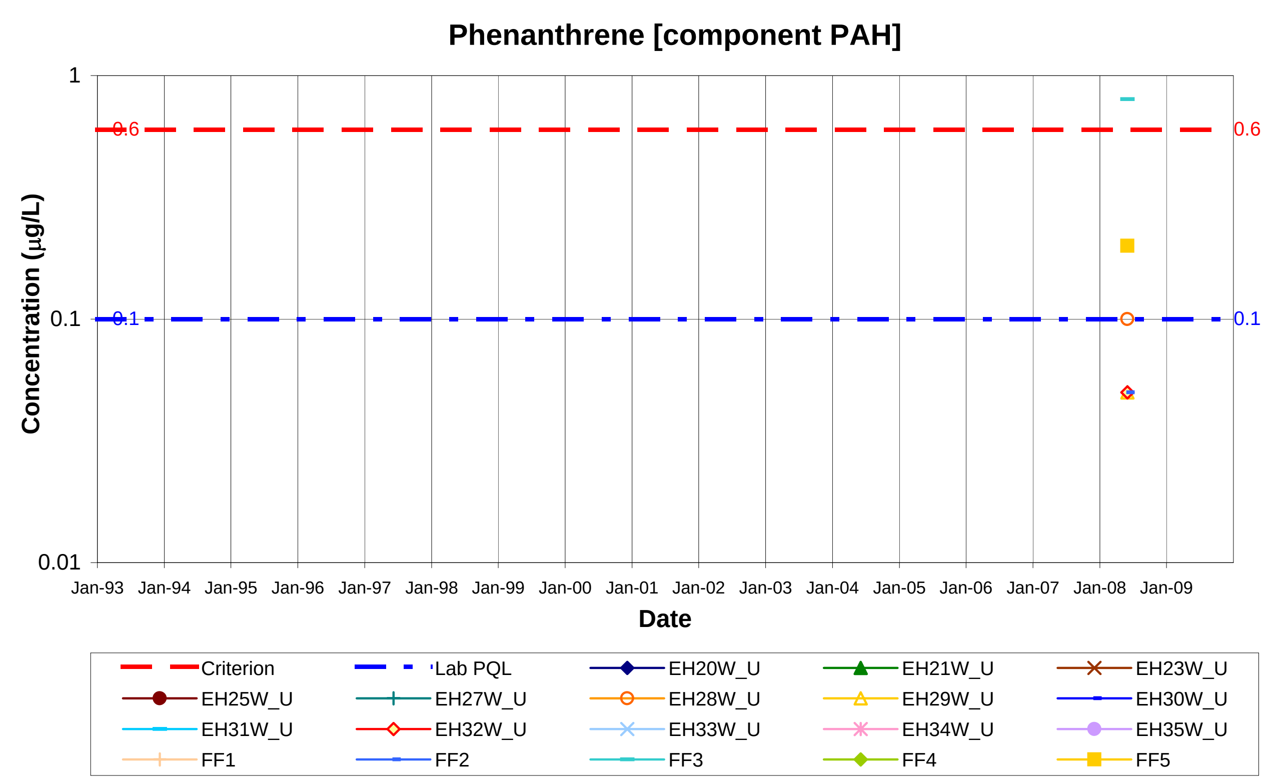

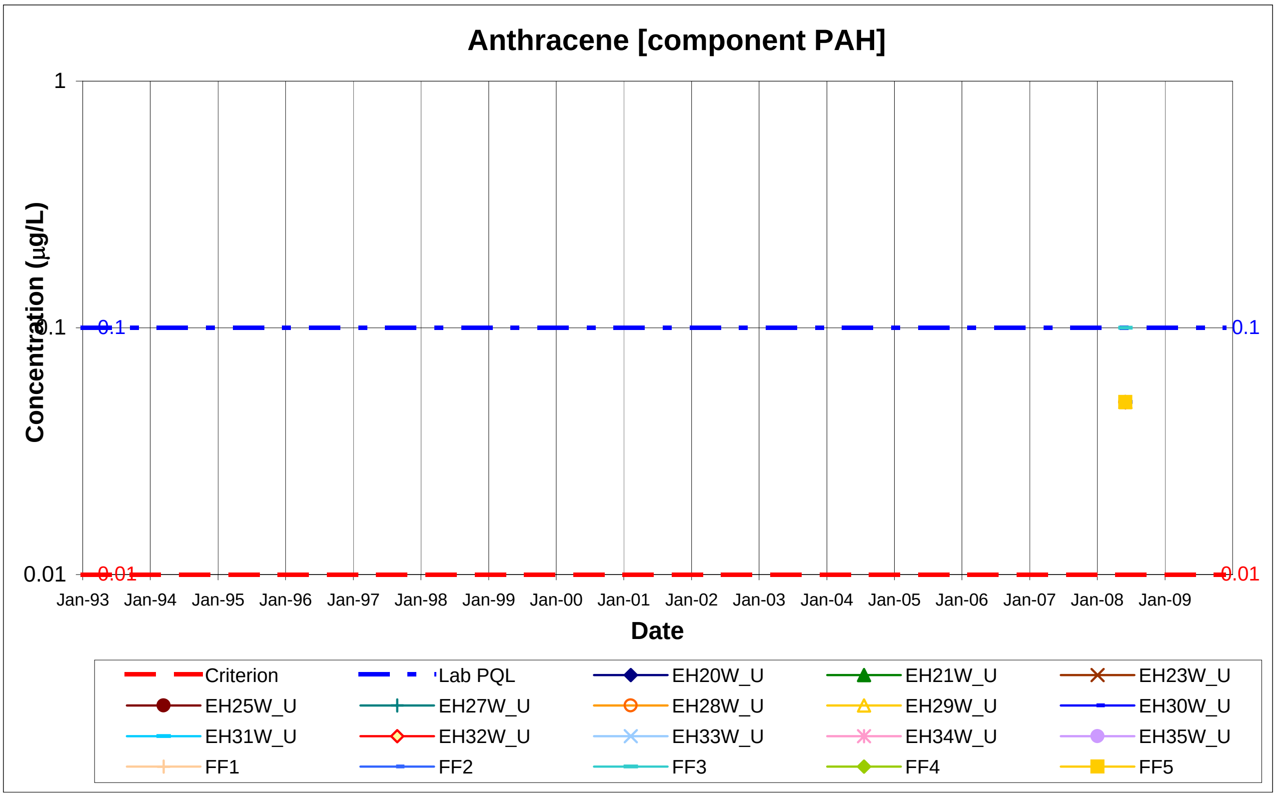

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PWCS Kooragang Coal Terminal

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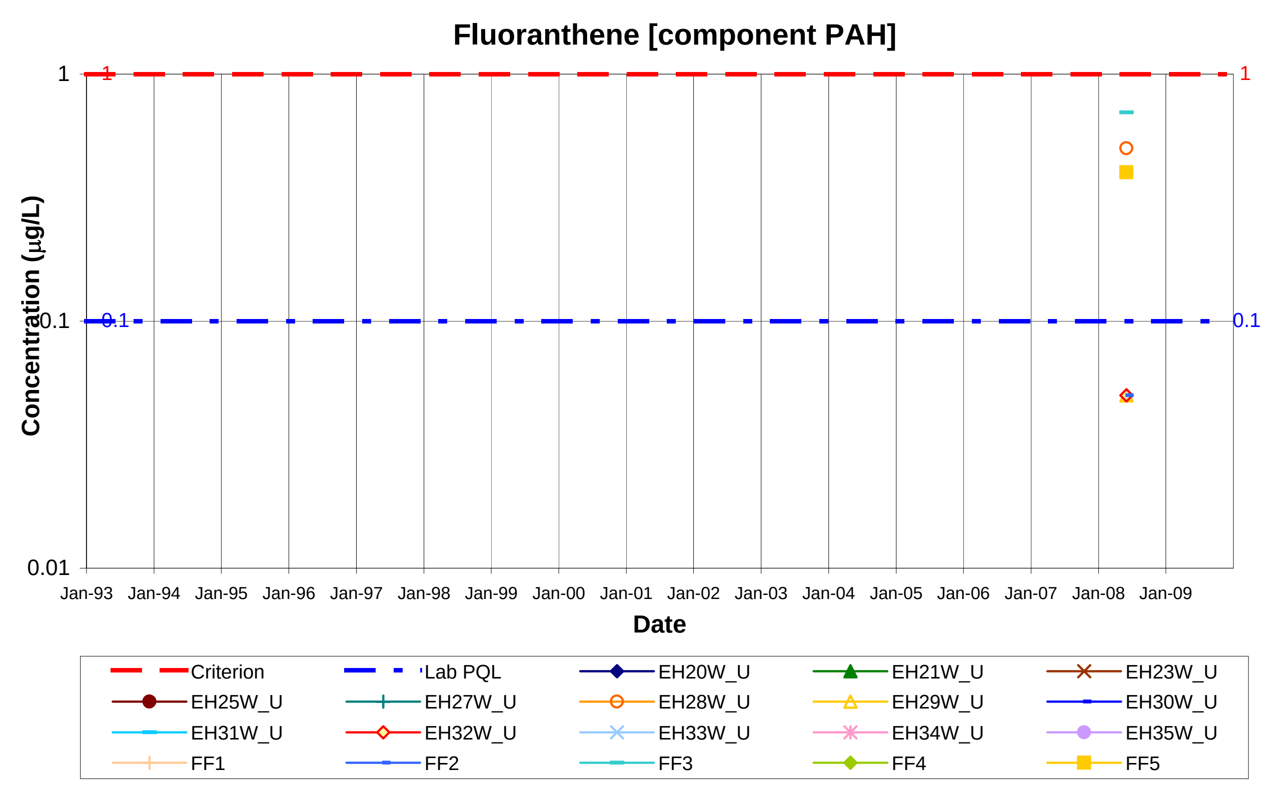

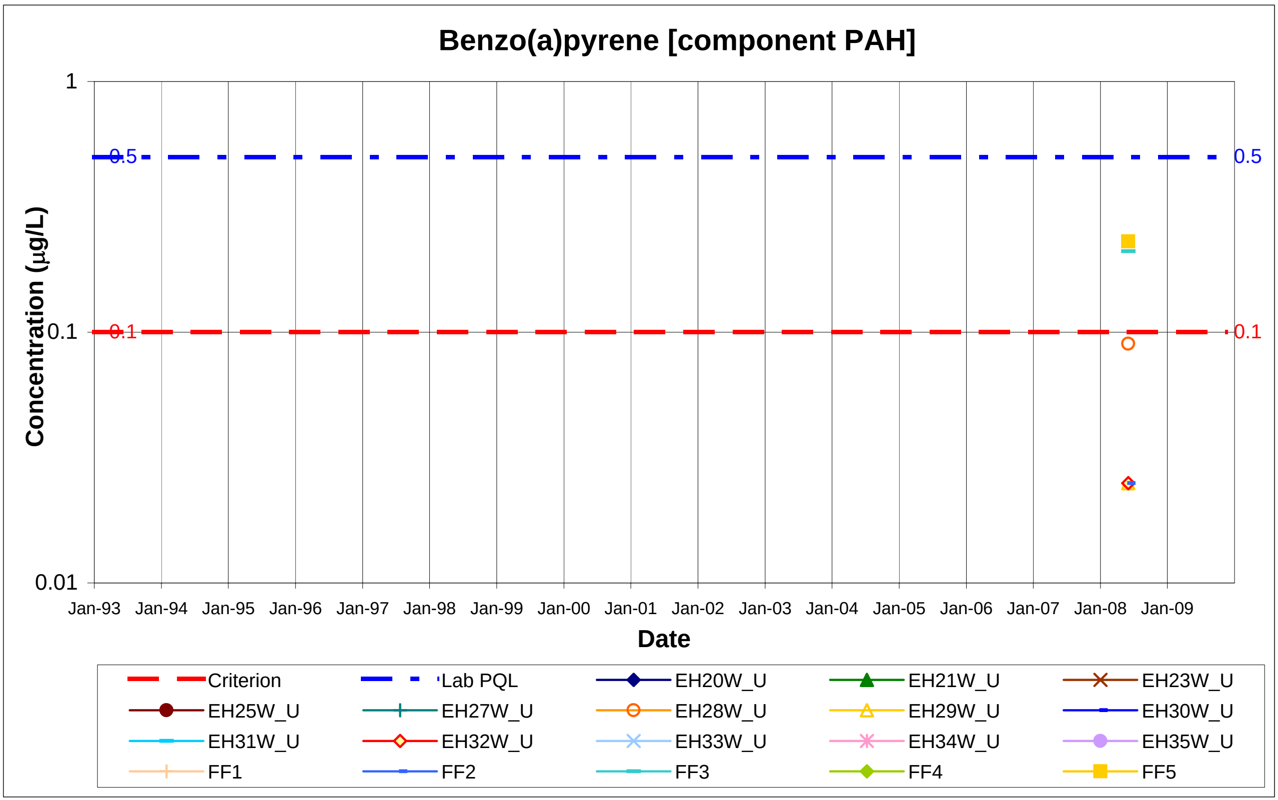

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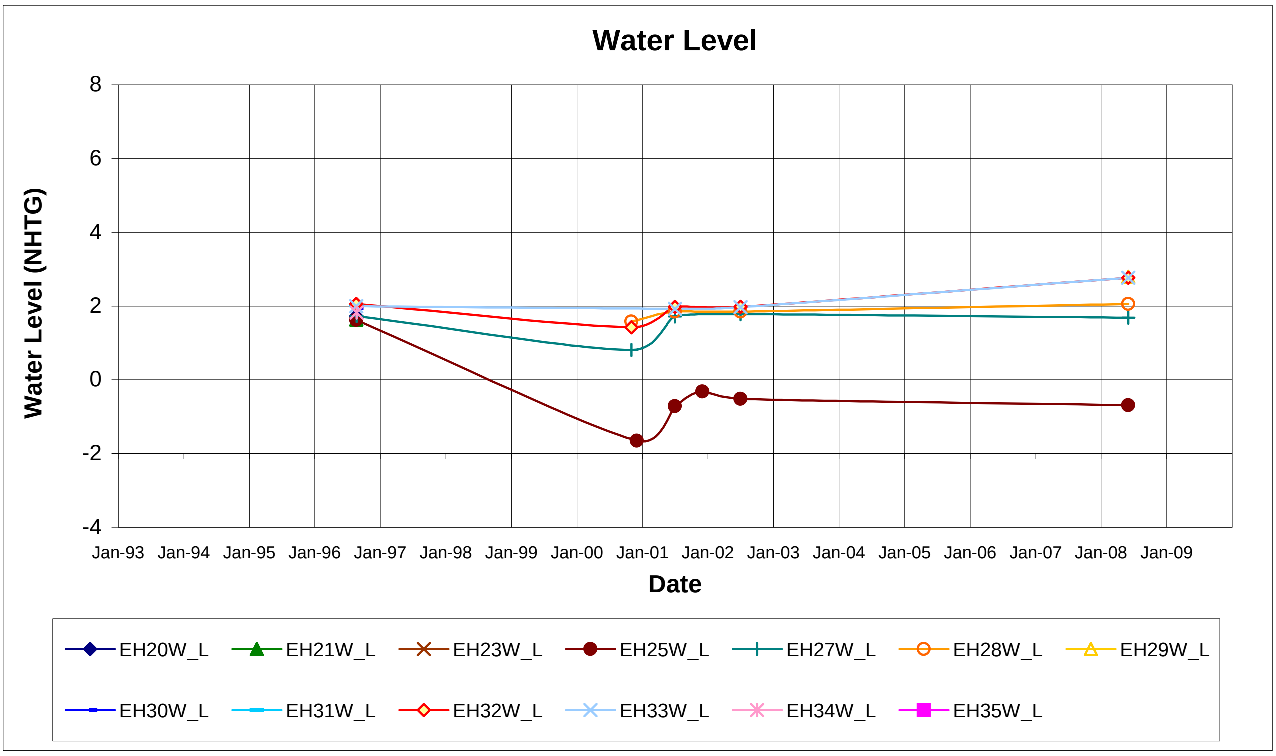
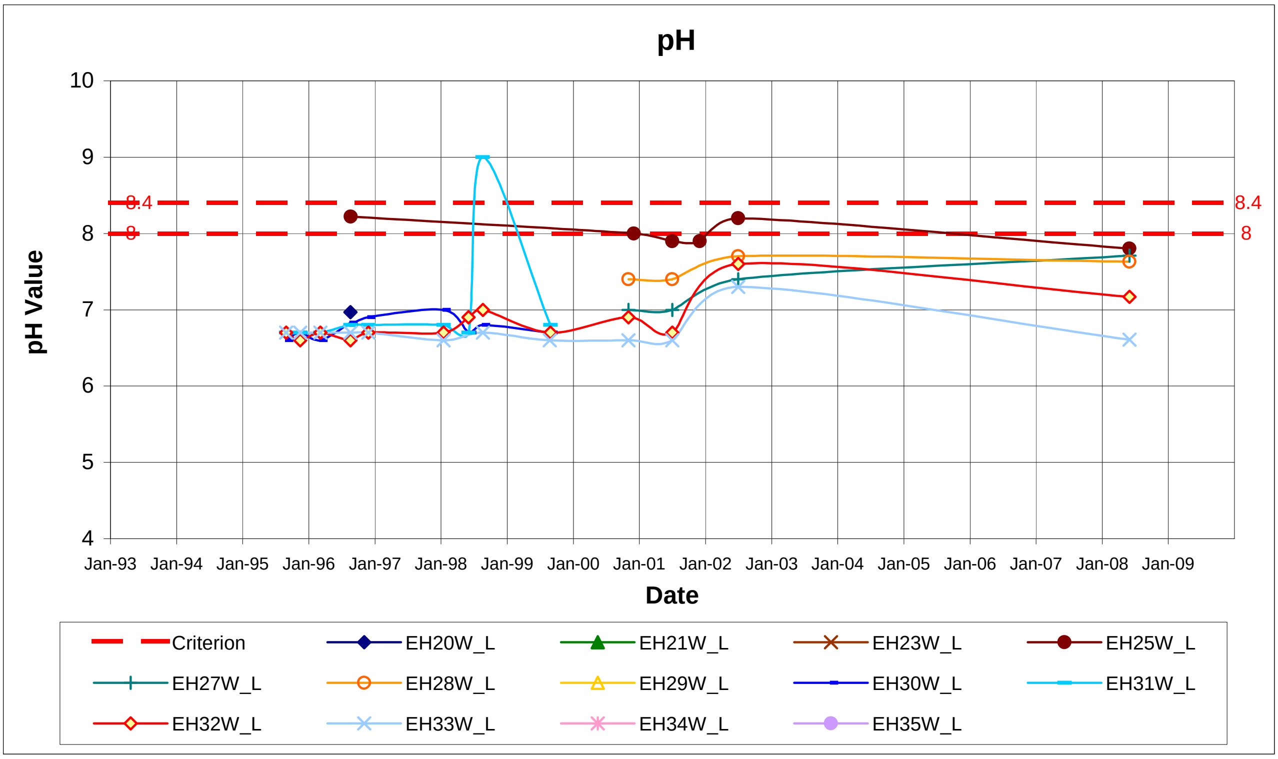
PWCS Kooragang Coal Terminal

GROUNDWATER QUALITY MONITORING

Aquifer: **Lower (Estuarine) Aquifer** **Project:** **49322**





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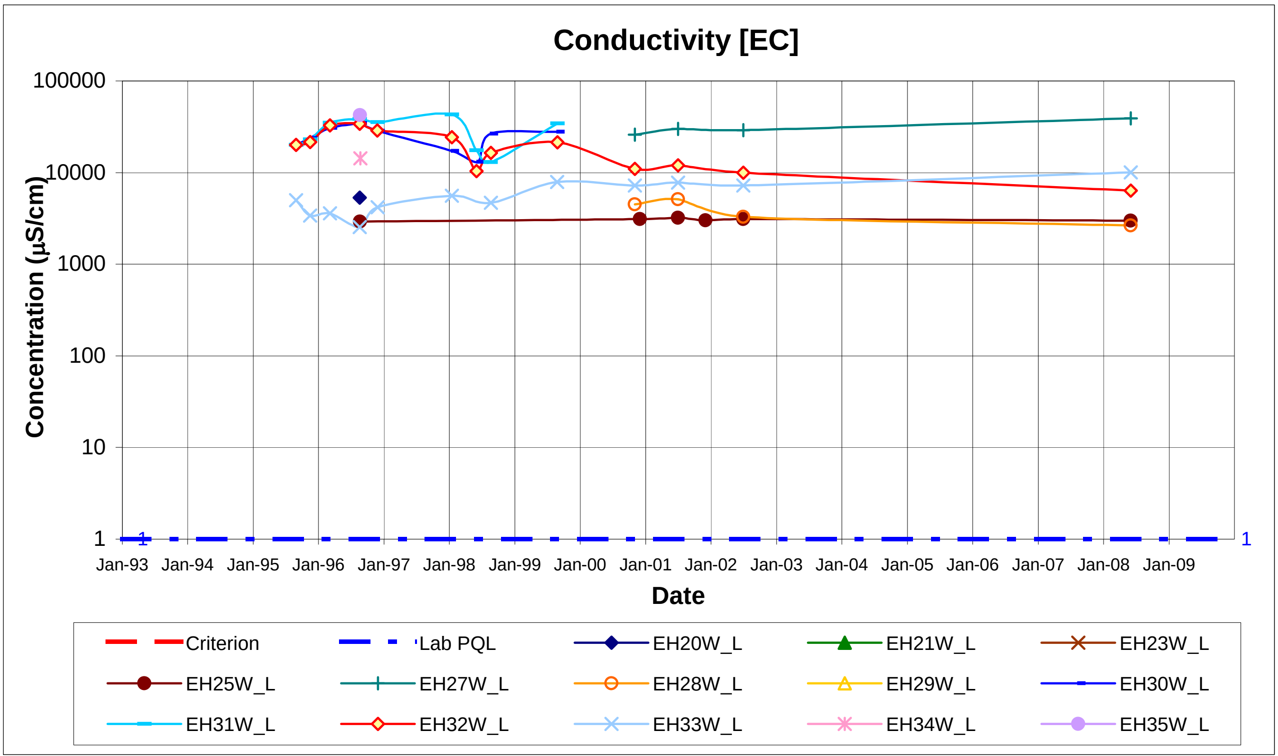
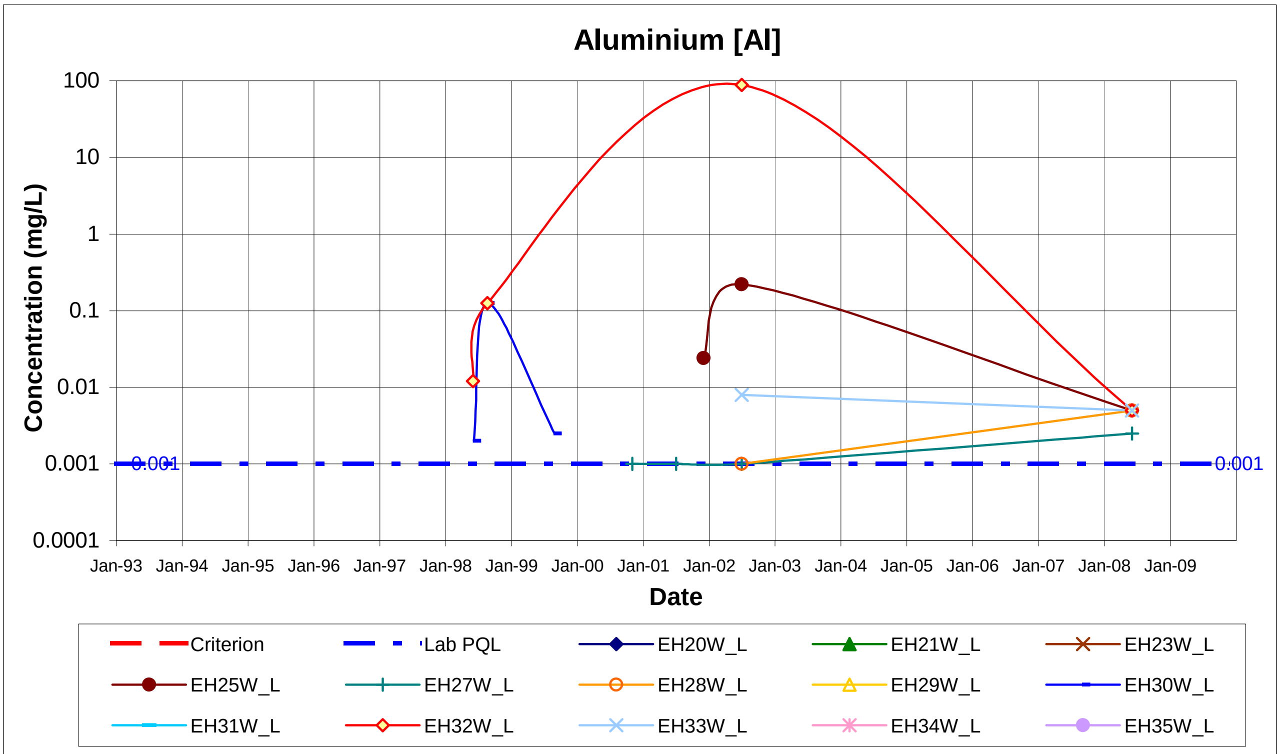
PWCS Kooragang Coal Terminal

GROUNDWATER QUALITY MONITORING

Aquifer: Lower (Estuarine) Aquifer **Project:** 49322





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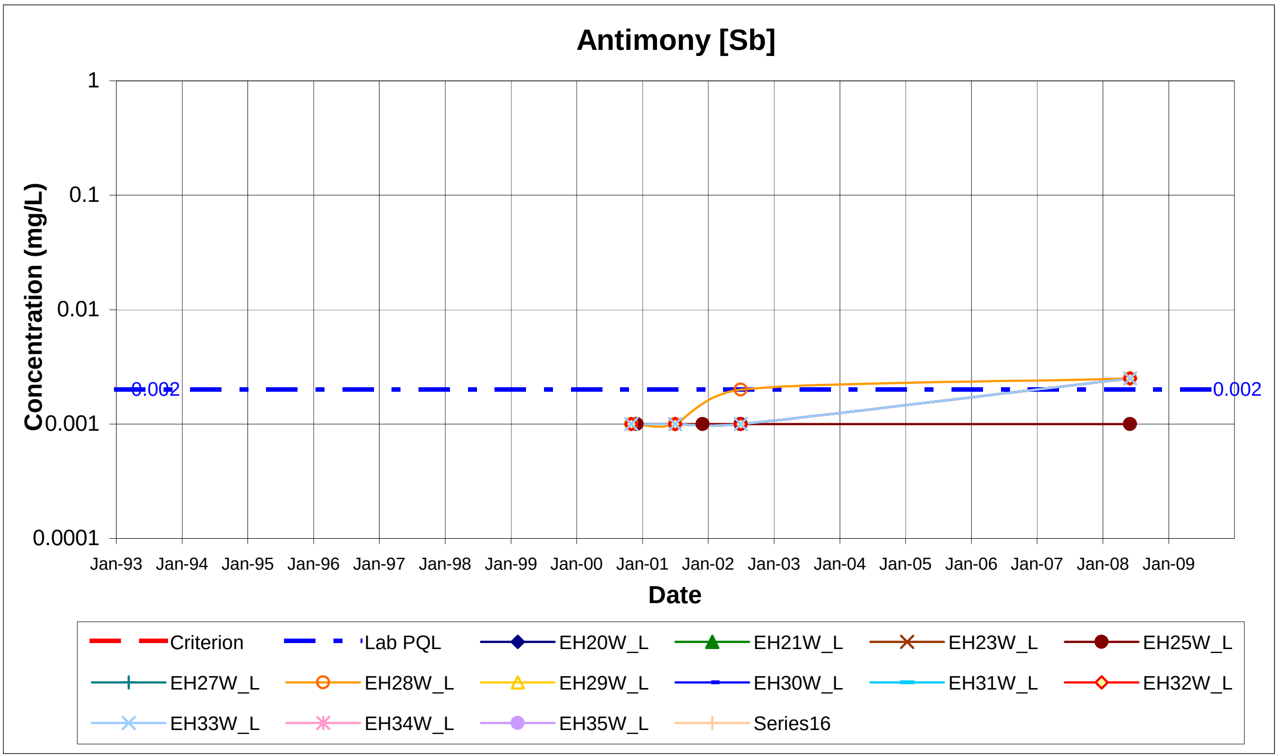

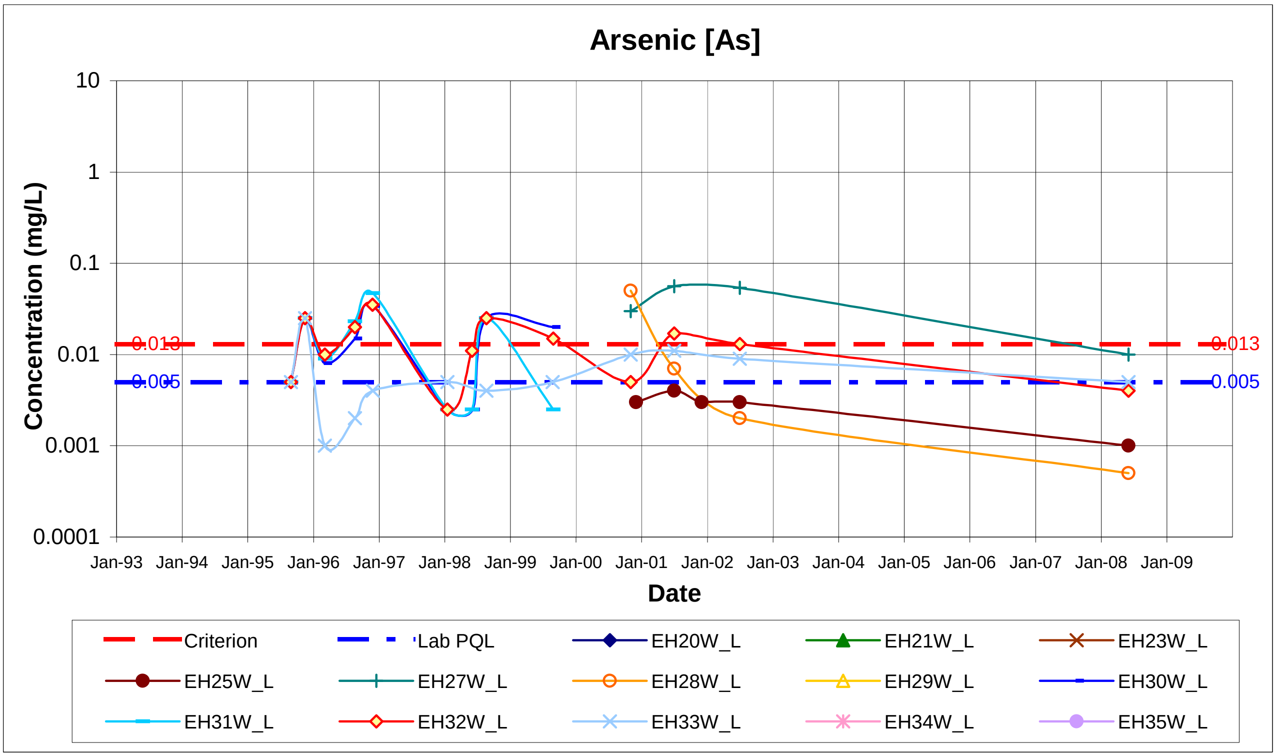

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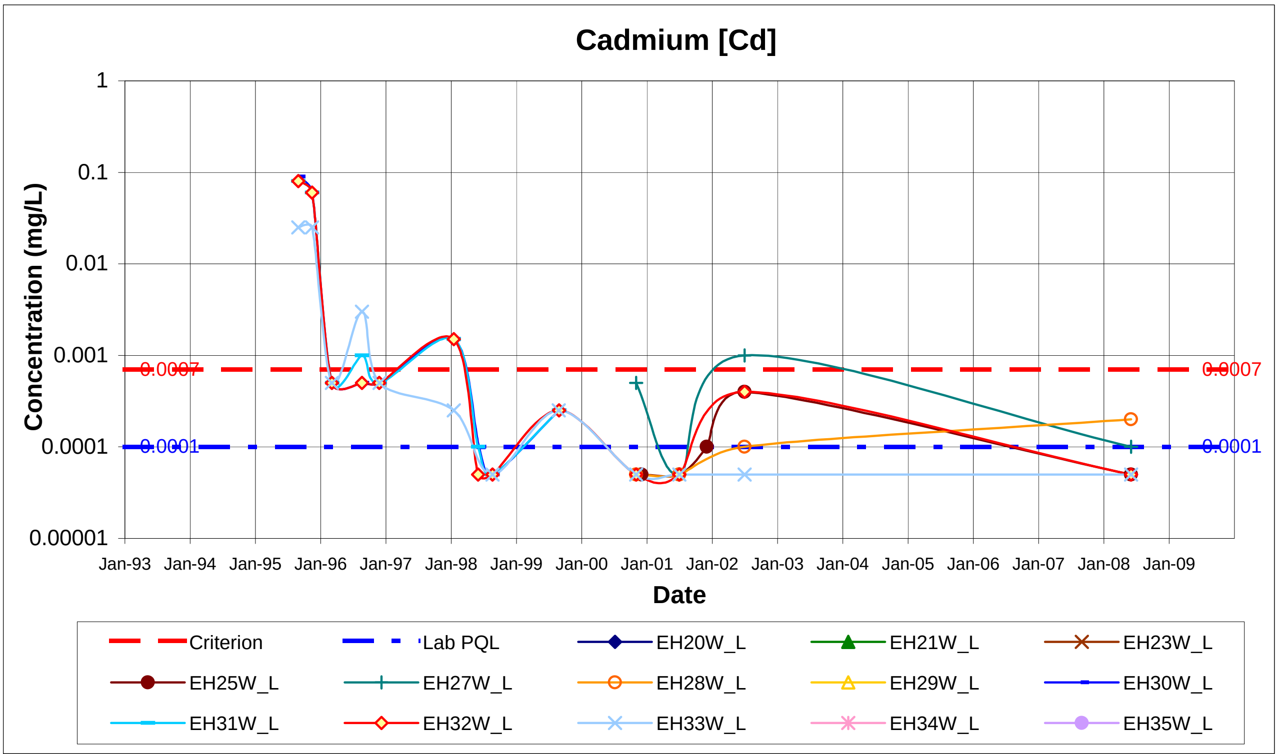

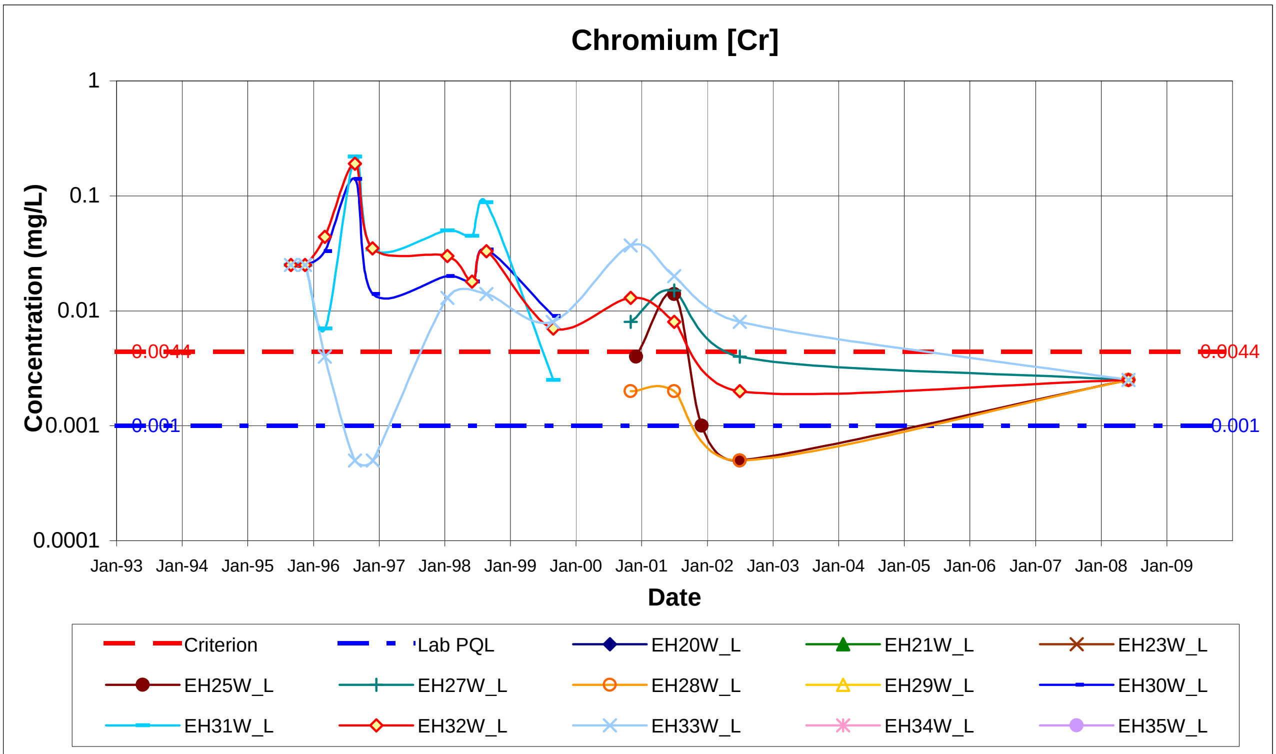

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PWCS Kooragang Coal Terminal
GROUNDWATER QUALITY MONITORING

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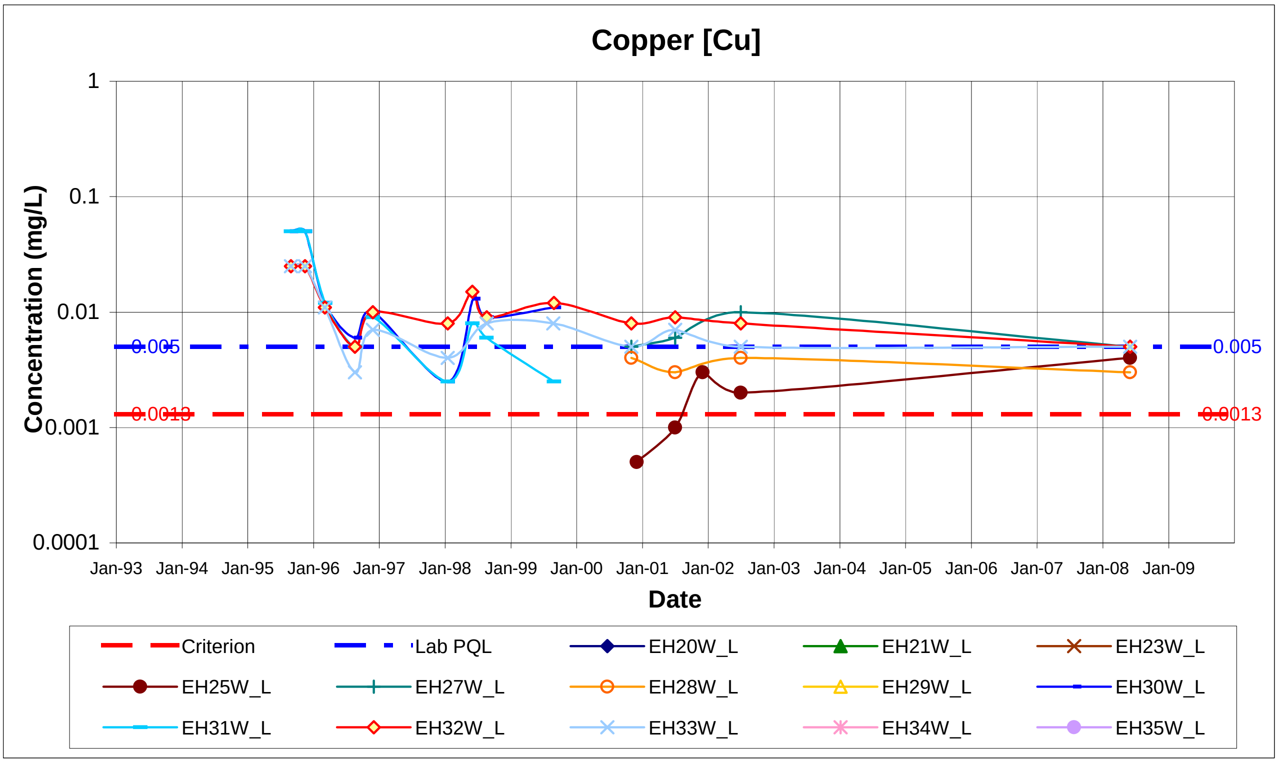

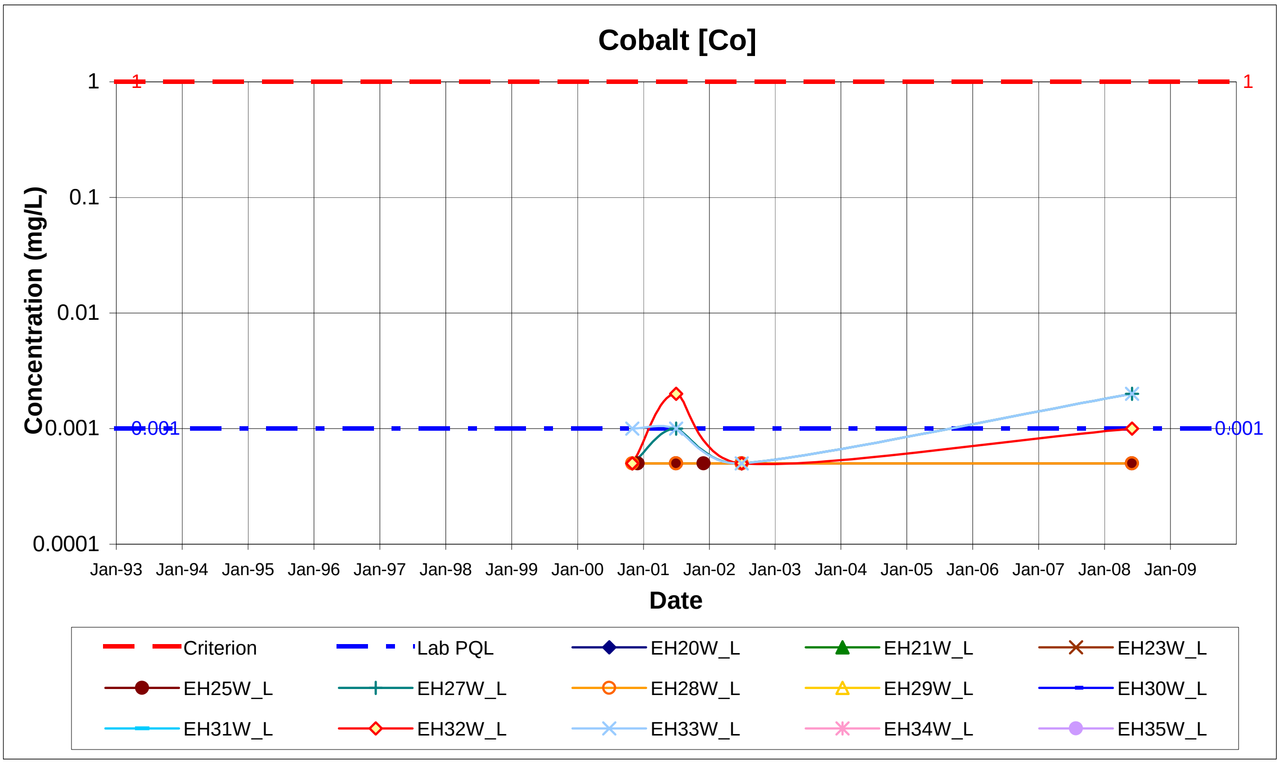

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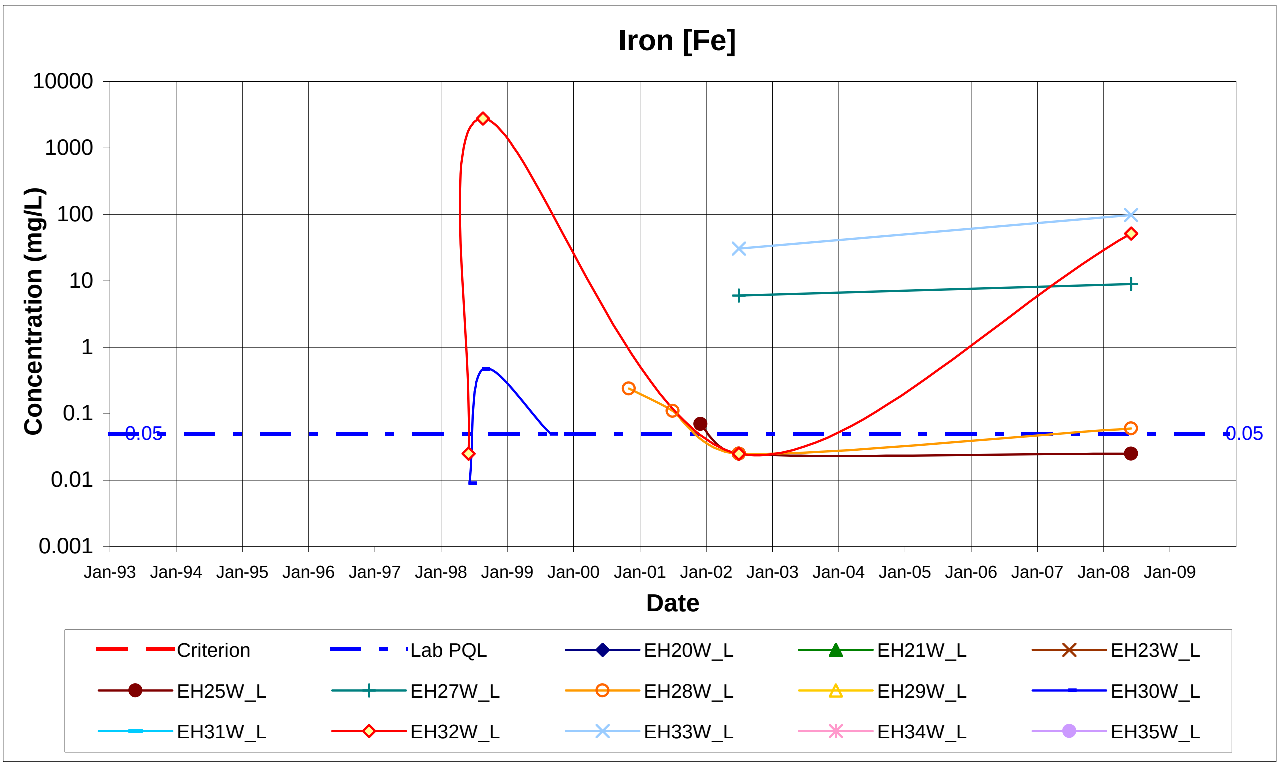

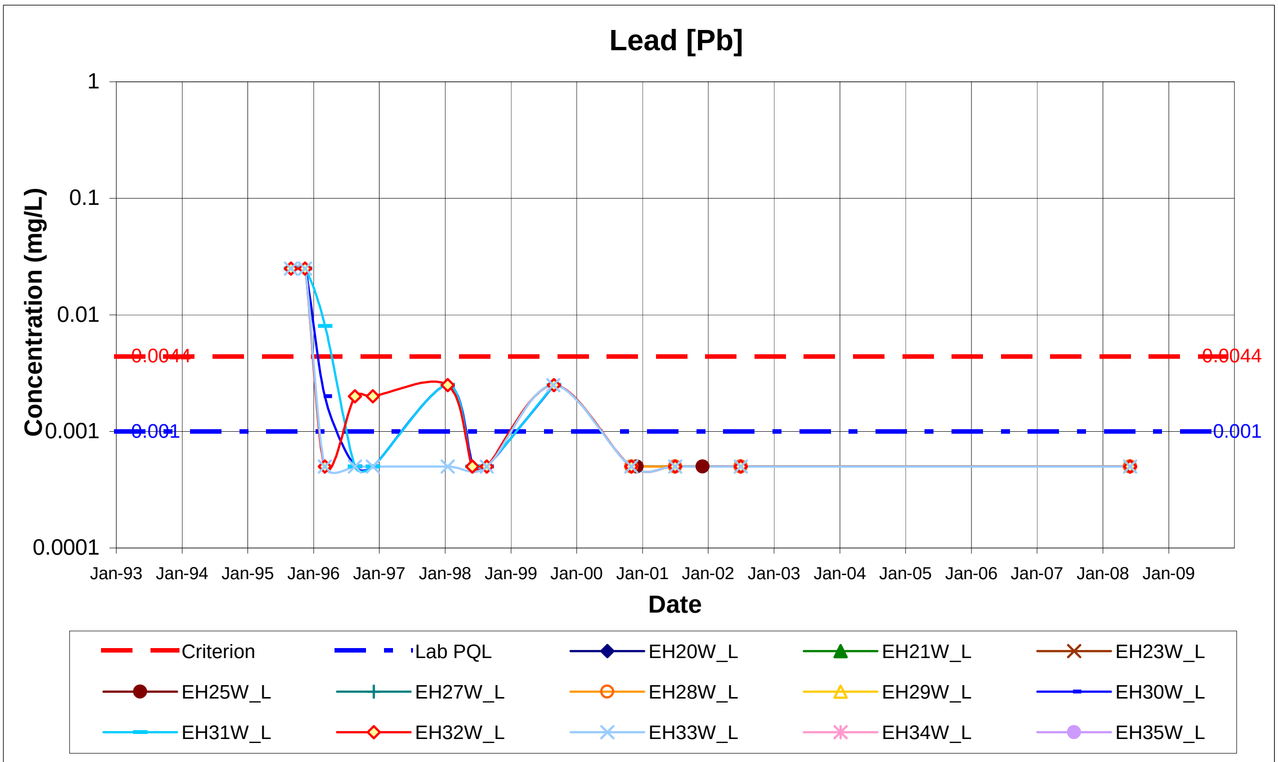

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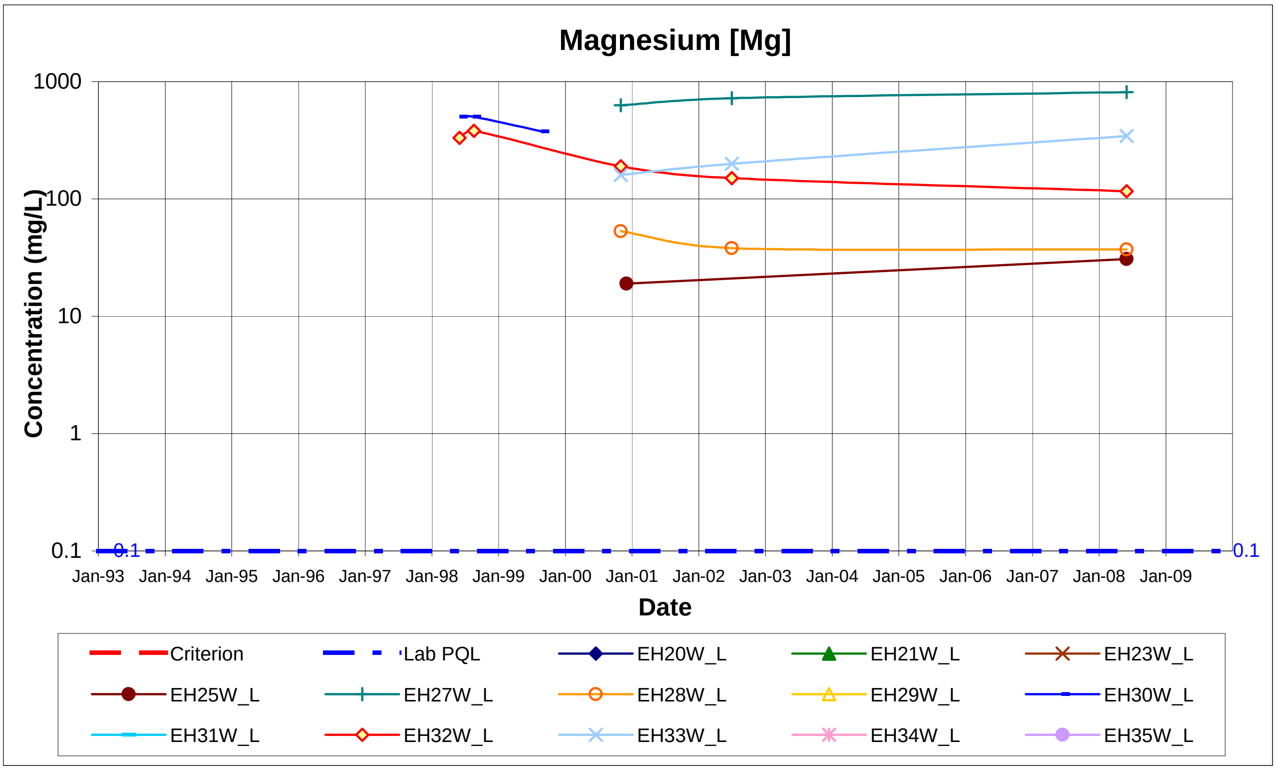

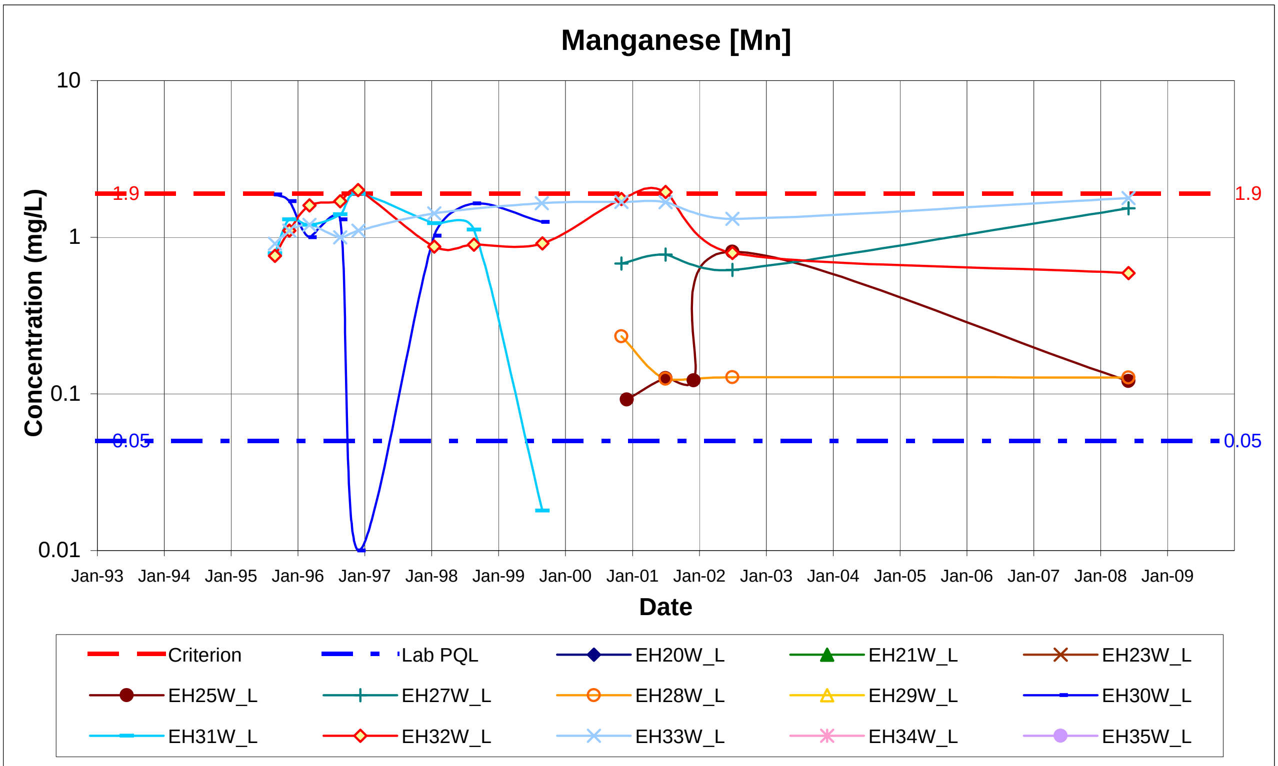

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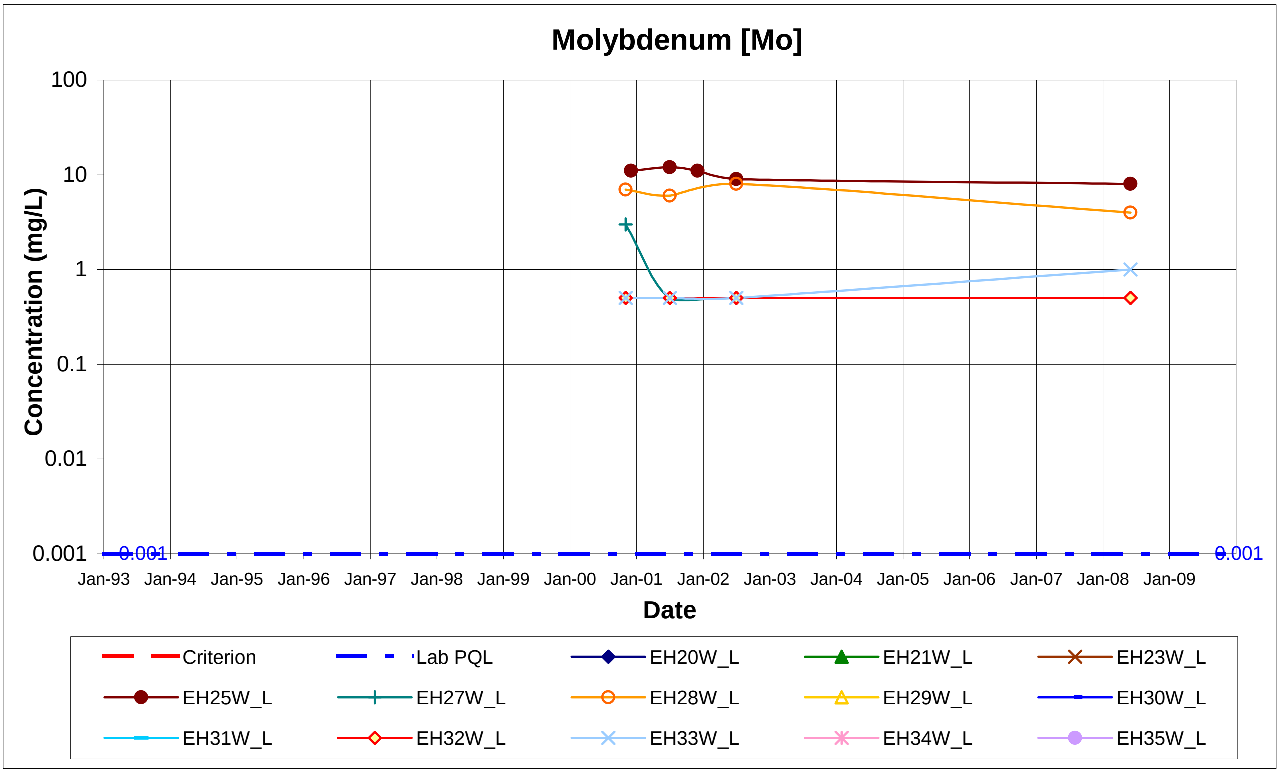

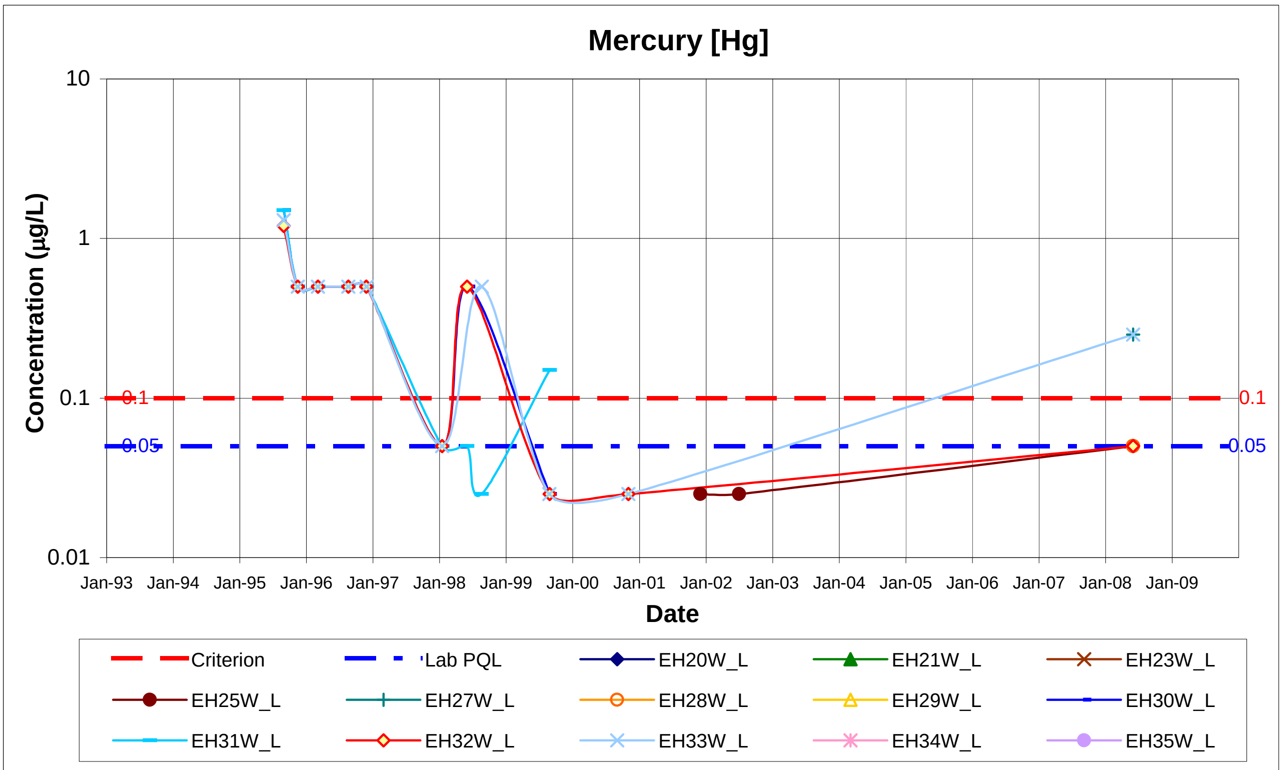

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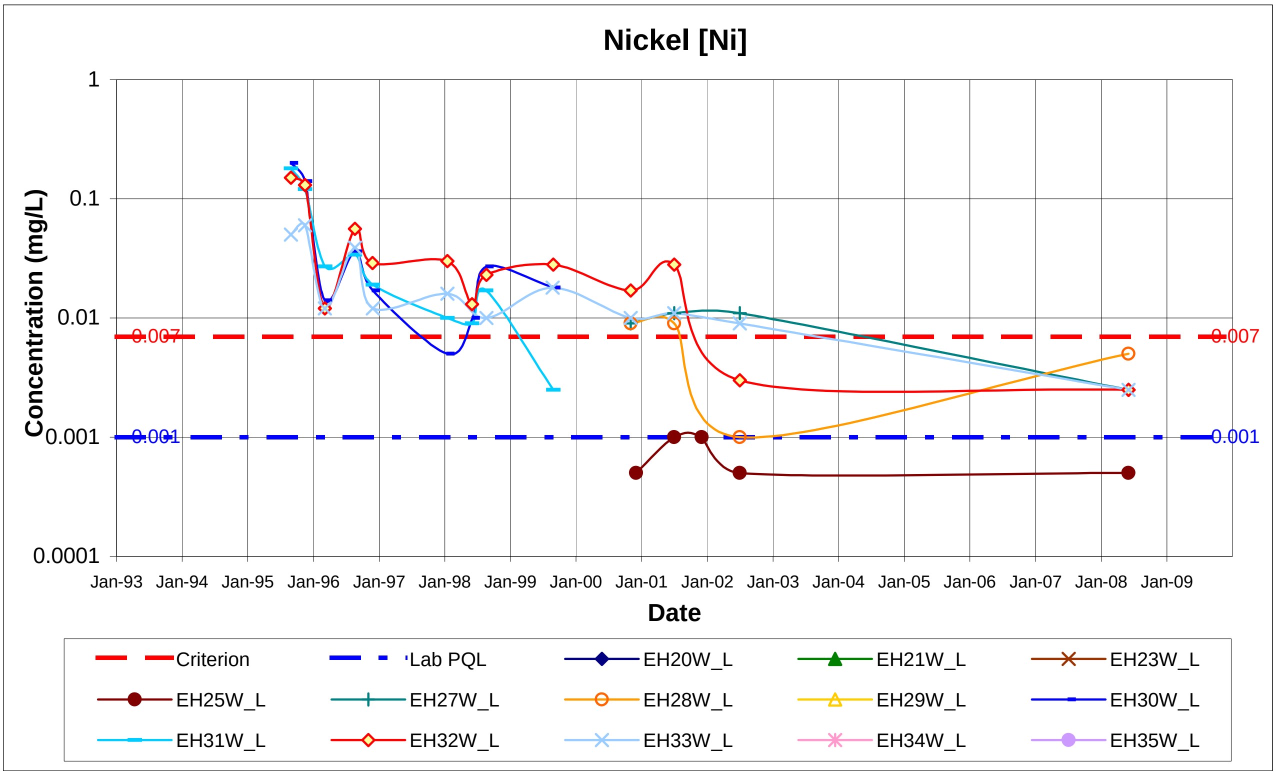

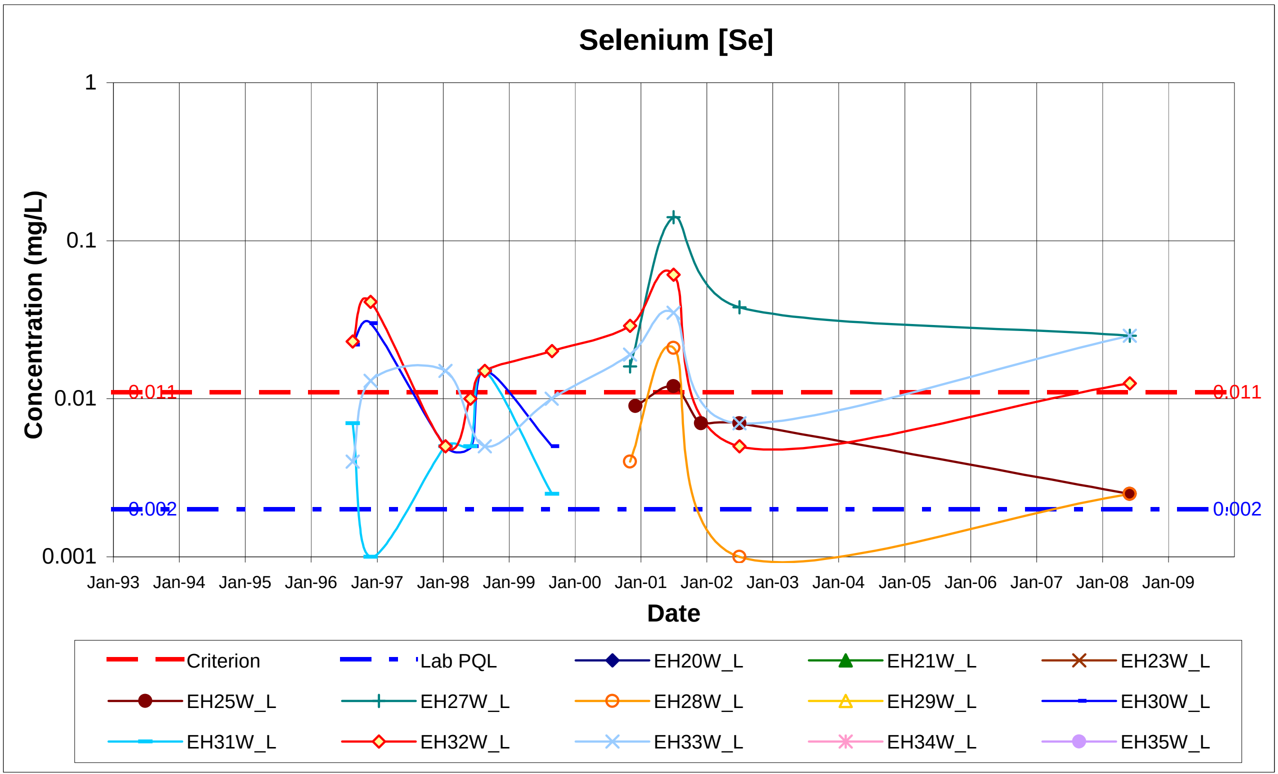

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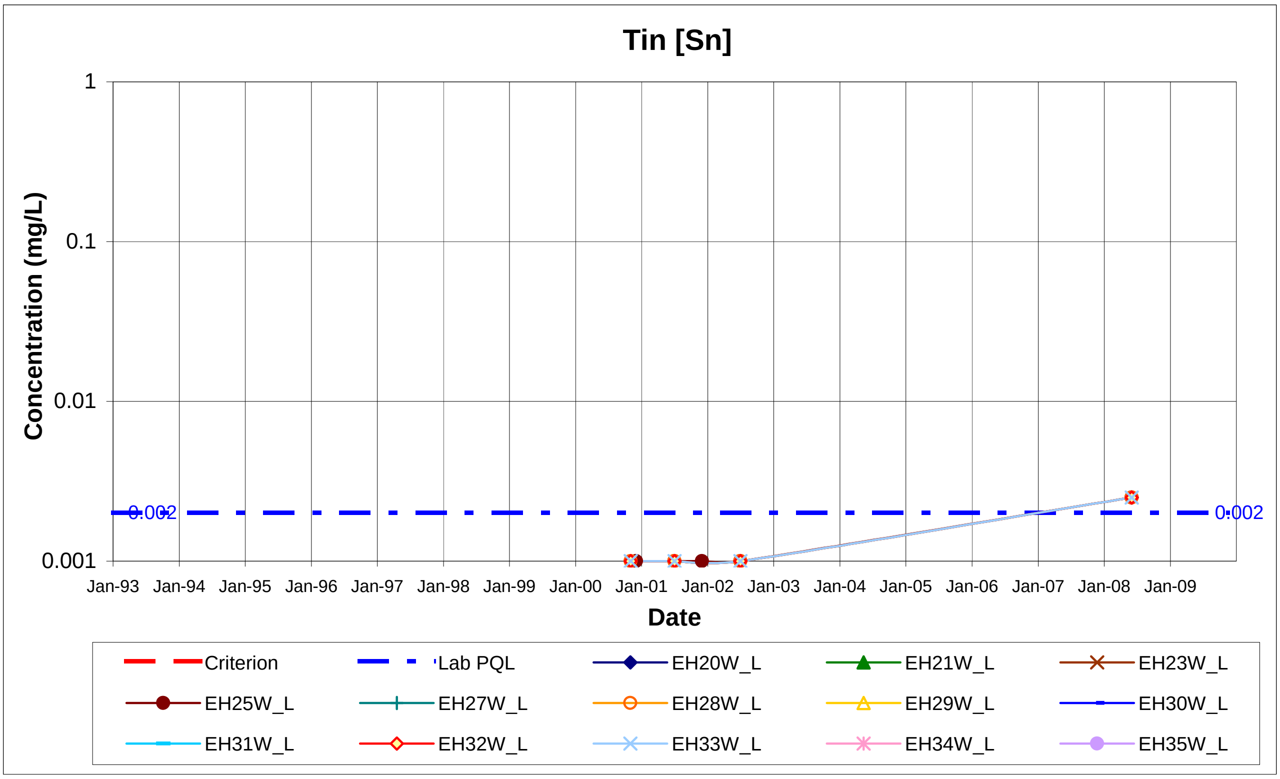
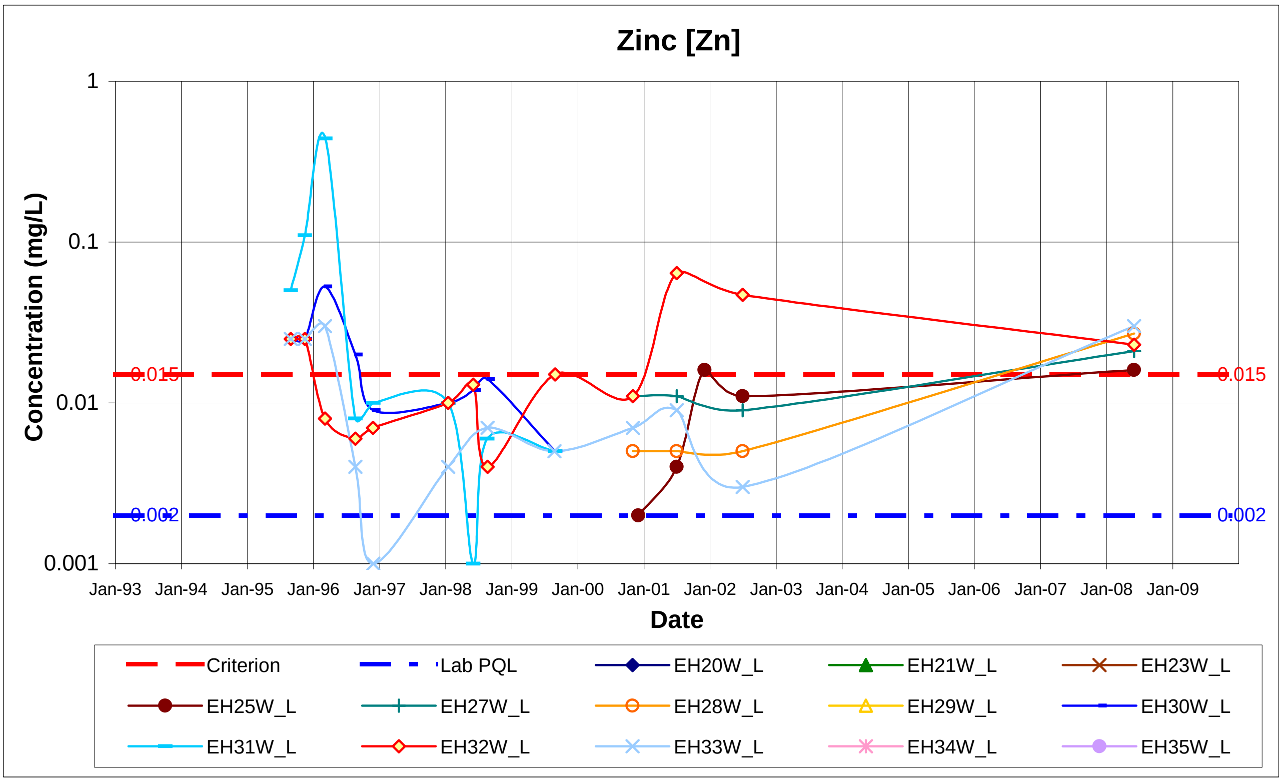
2. ANZECC 2000 Marine Criteria adopted; Fresh Water criteria adopted if no Marine value.
3. If a Criterion line is not plotted, there are No Criteria for the parameter.

PWCS Kooragang Coal Terminal

GROUNDWATER QUALITY MONITORING

Aquifer: Lower (Estuarine) Aquifer **Project:** 49322





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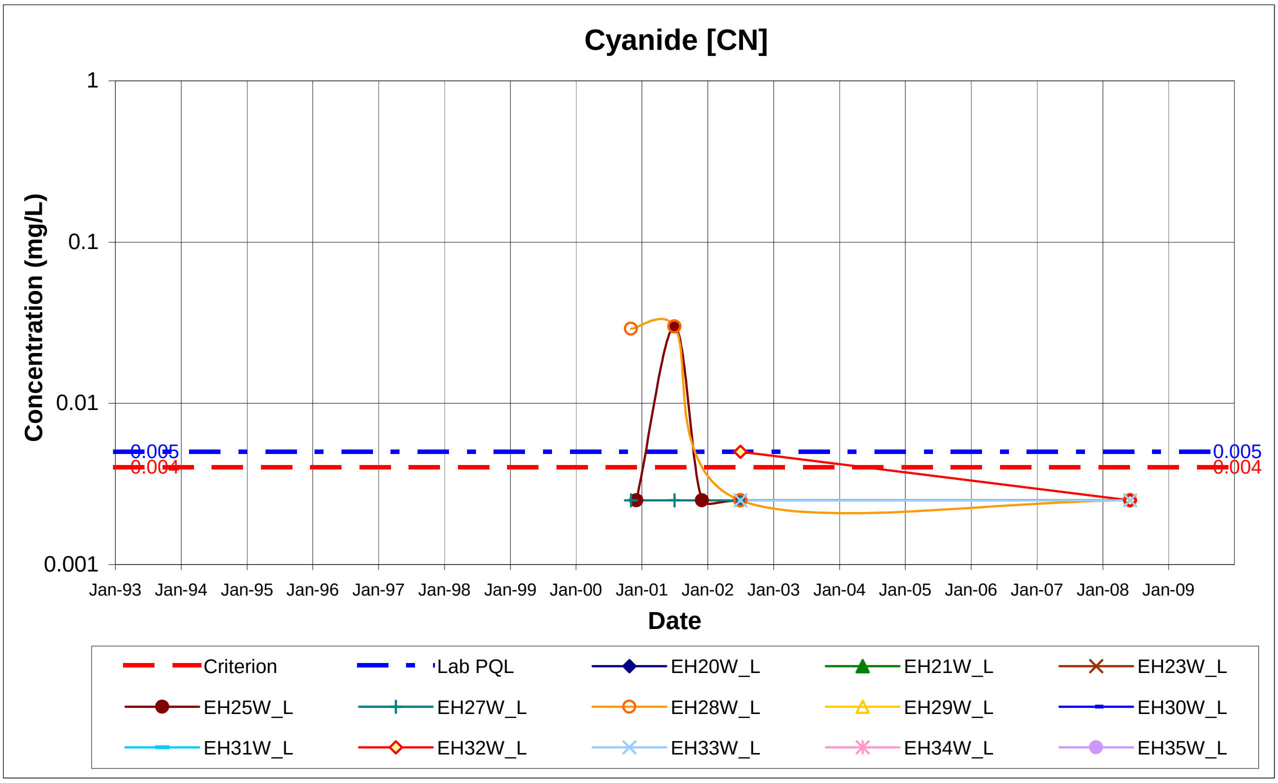

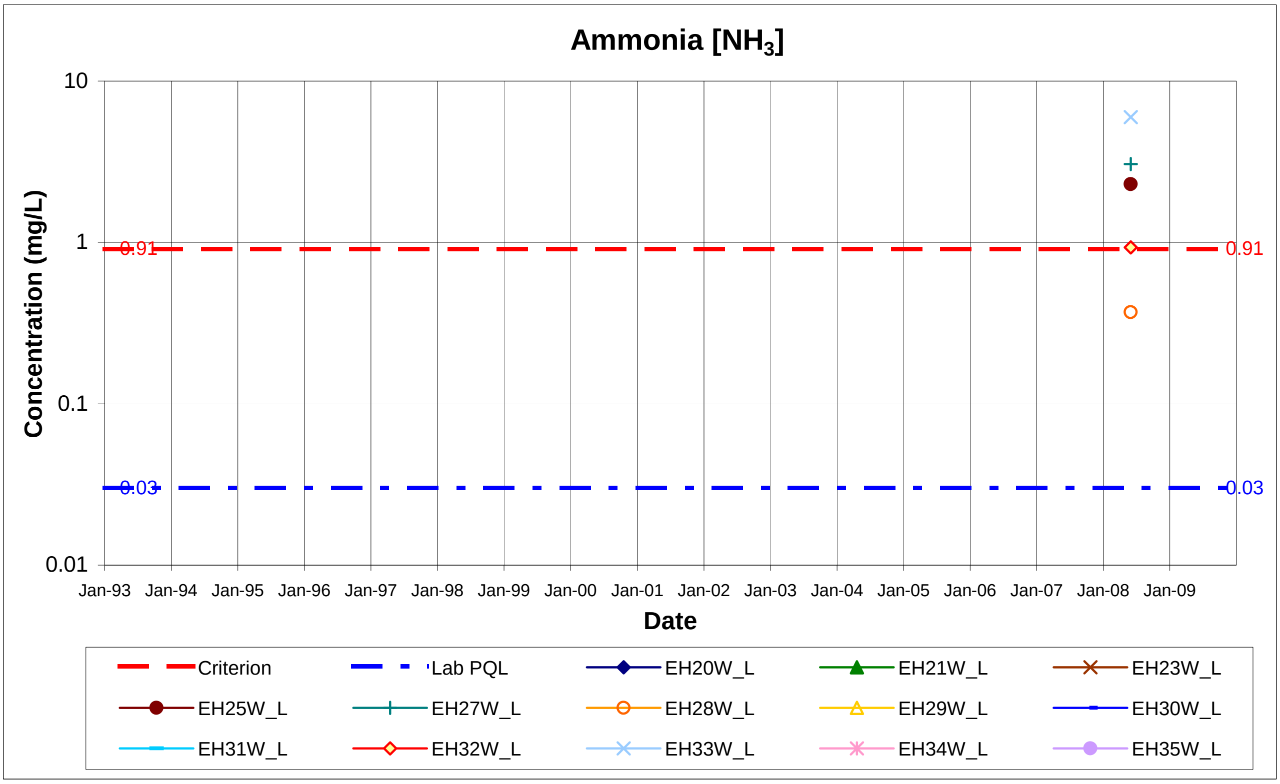

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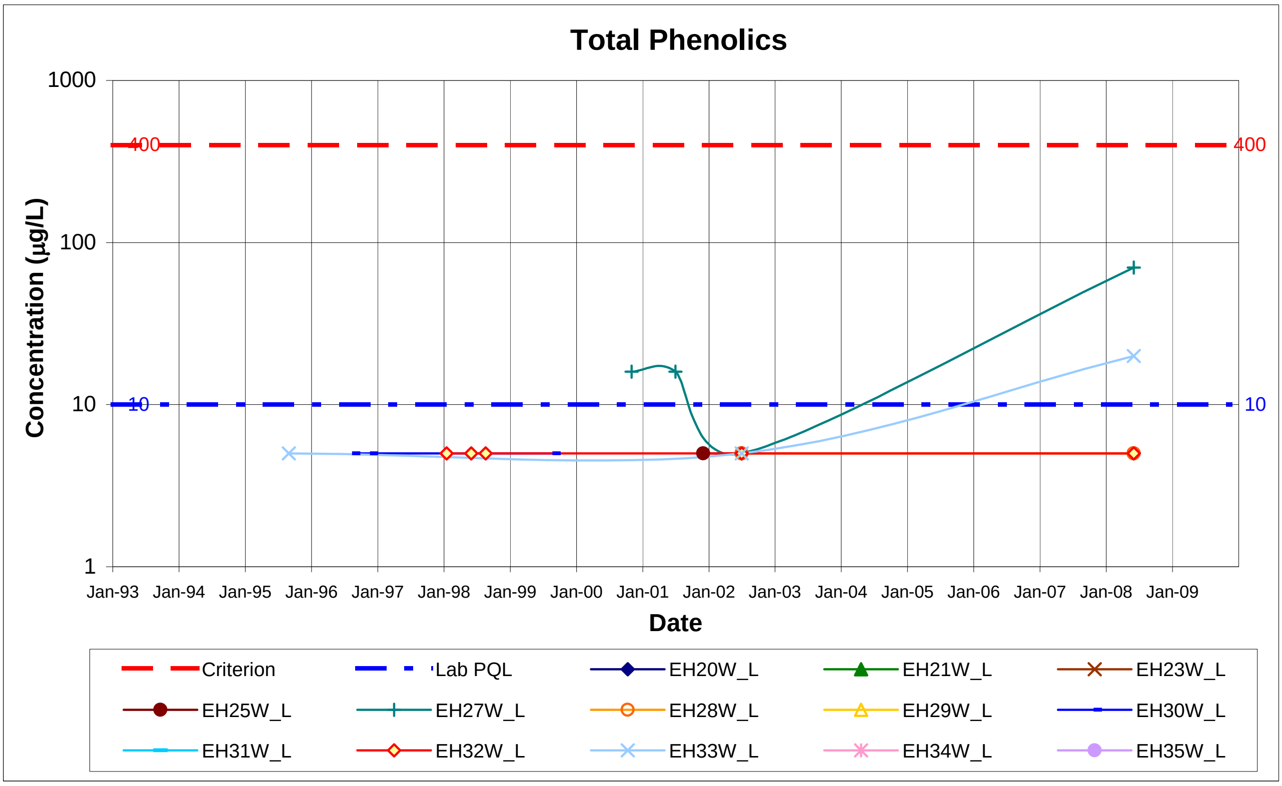

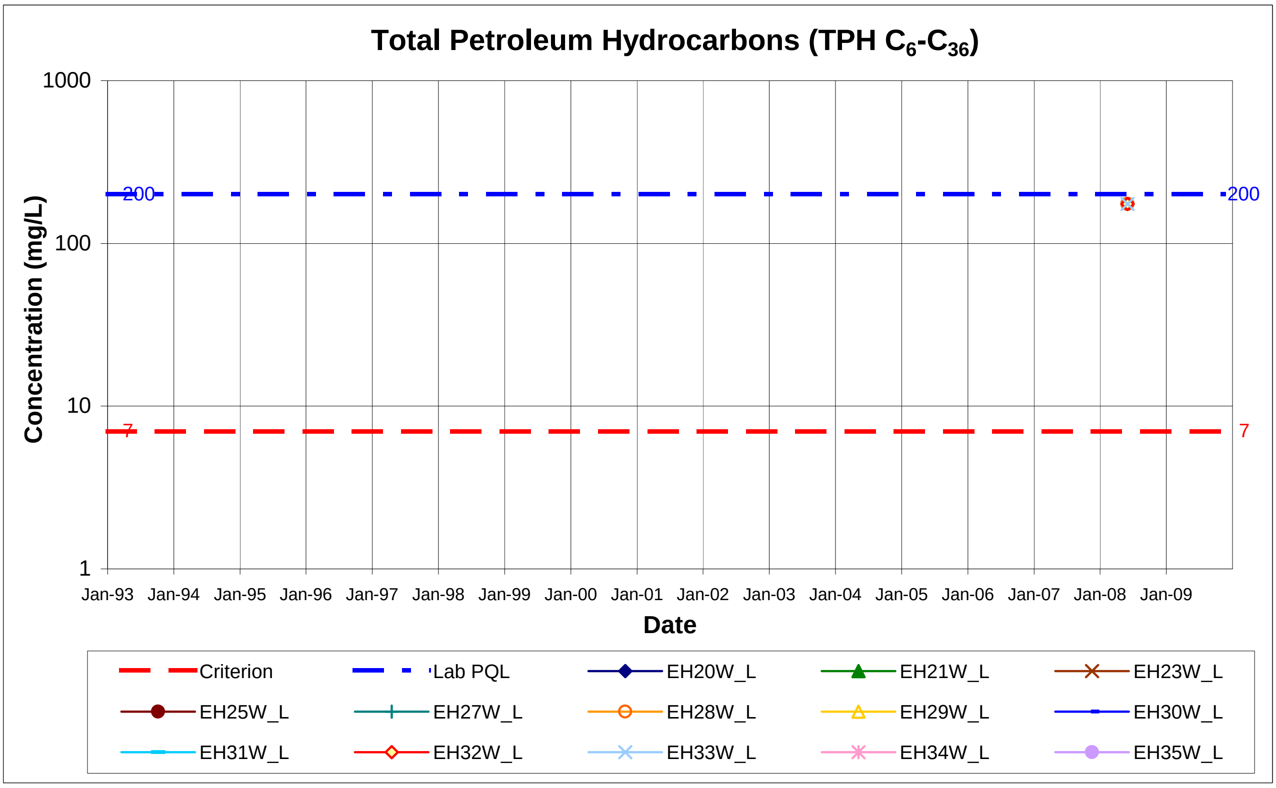

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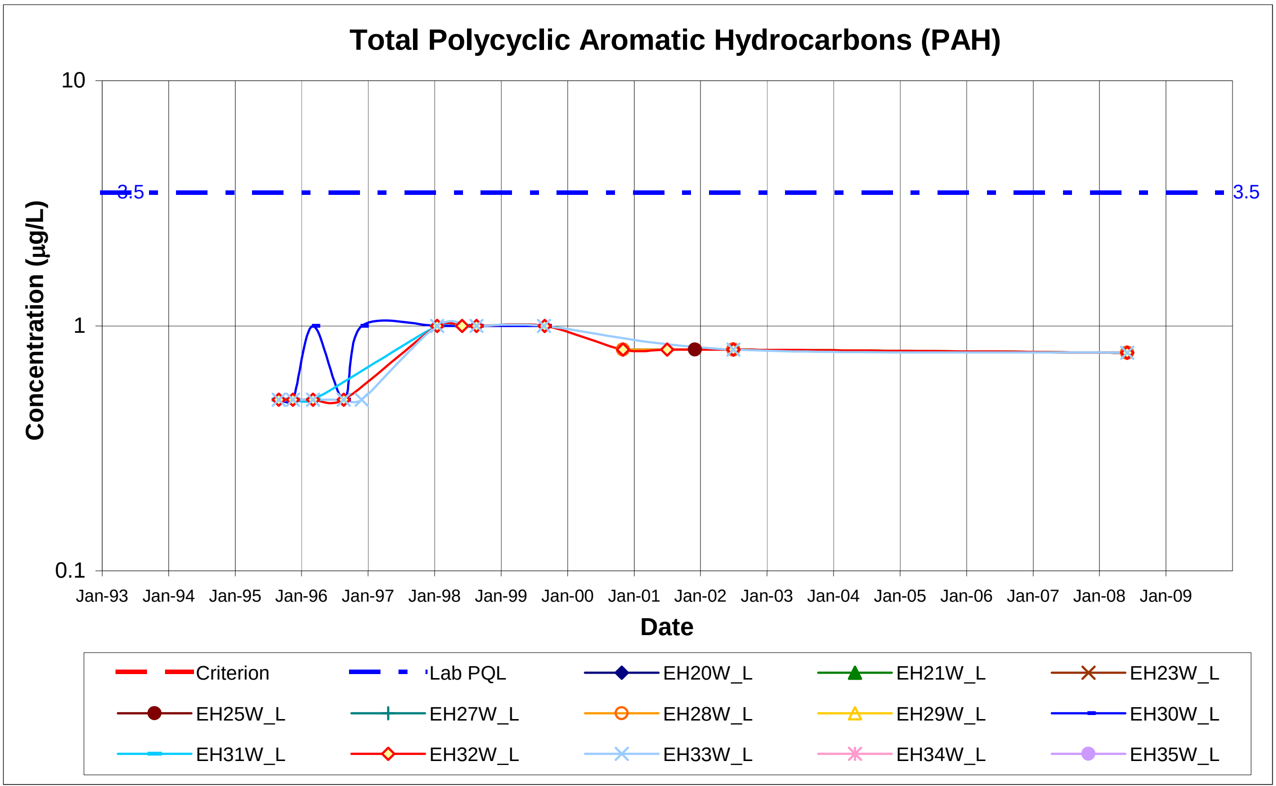

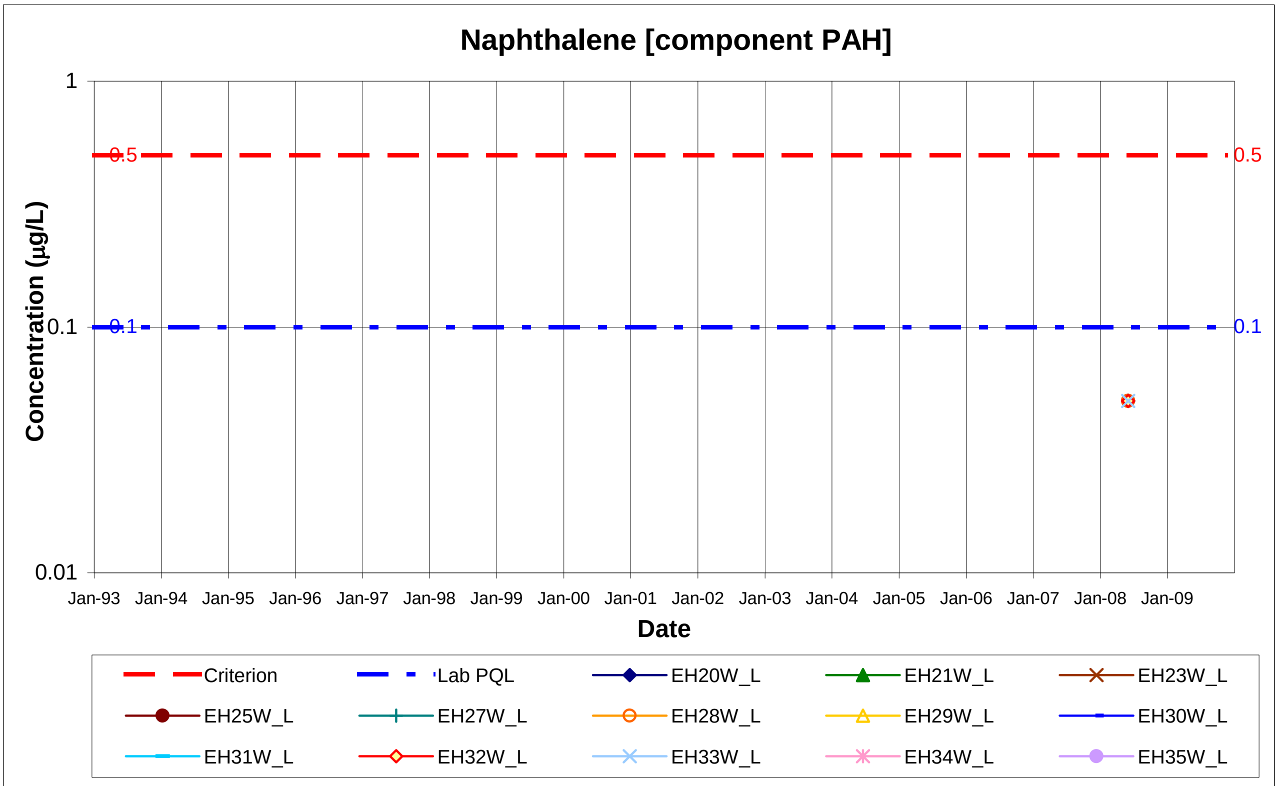

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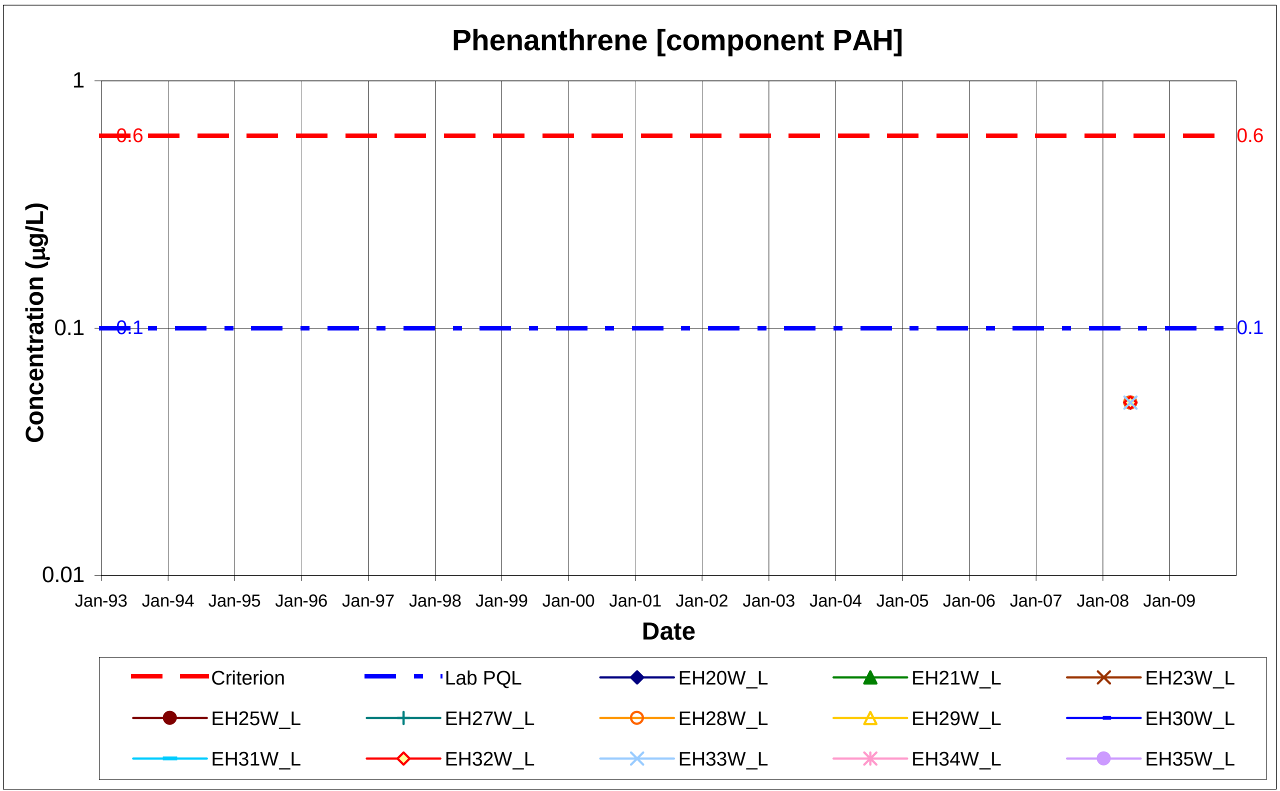

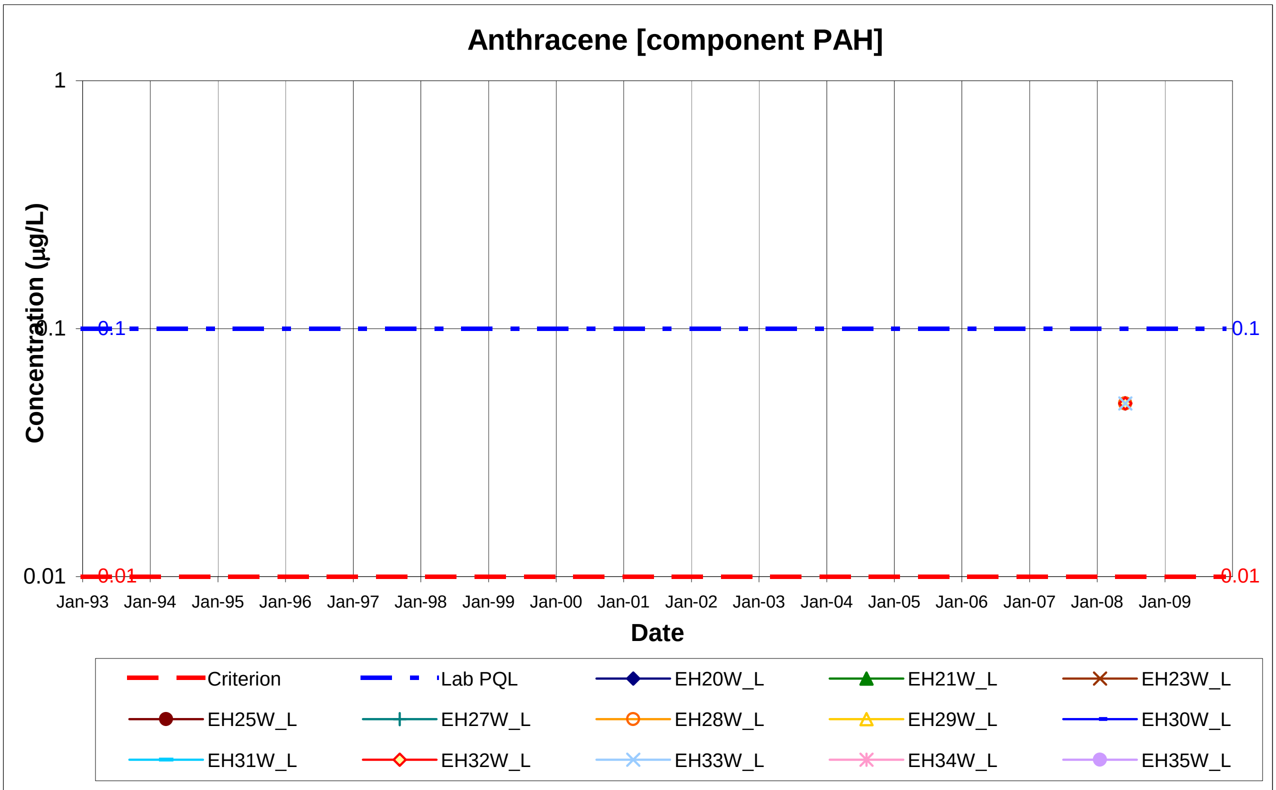

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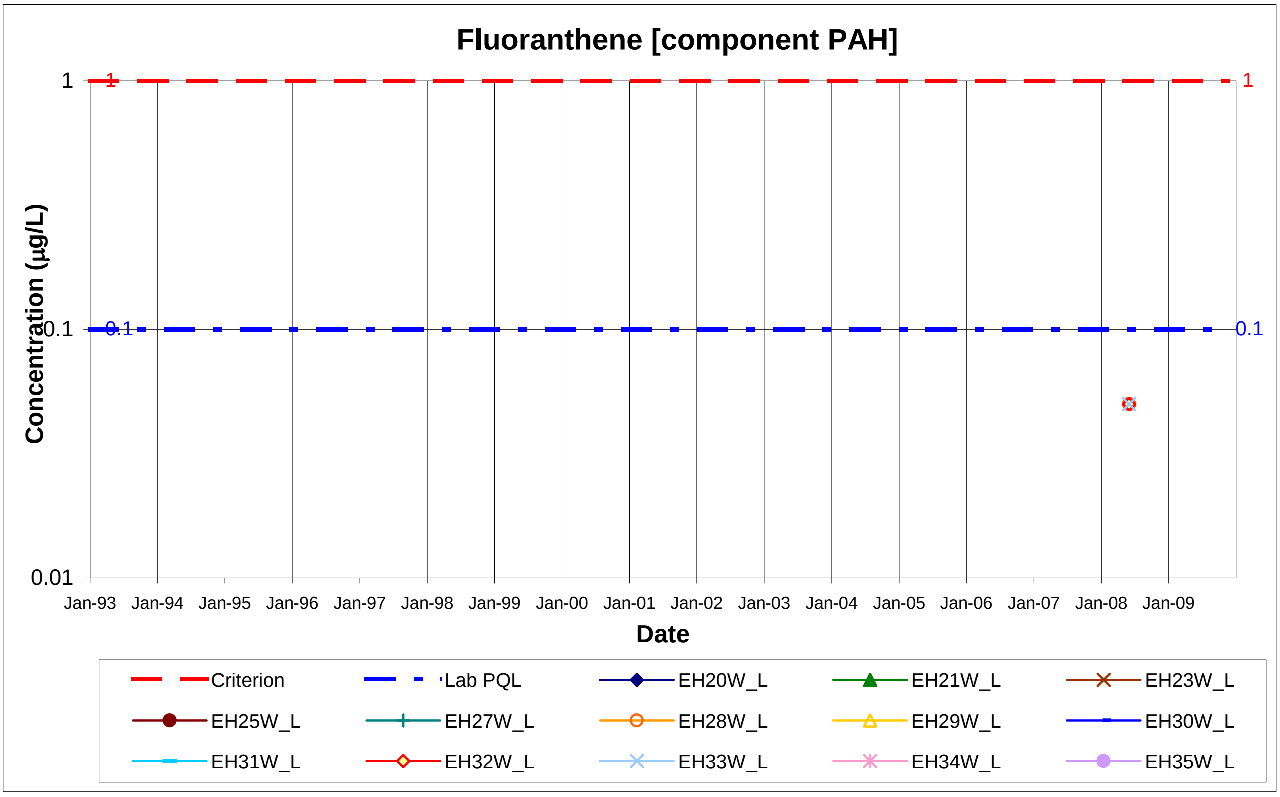

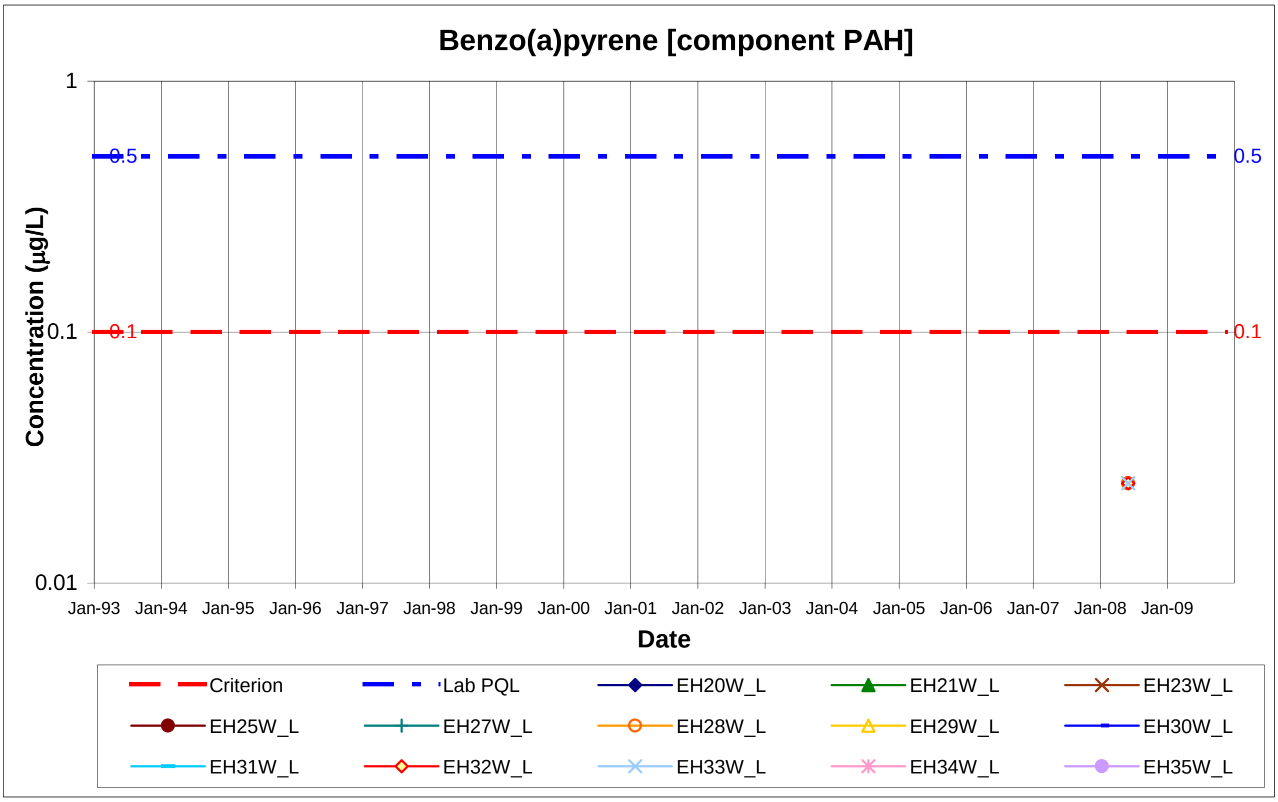

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Geotechnical Construction
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Project Summary

Jet Grouting
Soil Anchors
Slurry Wall

Cedar Bay Cogeneration Plant Jacksonville, Florida

Installation of a 173 ft by 53 ft coal unloading structure required a 31-ft deep excavation through shallow, potentially contaminated groundwater. Both dewatering of the excavation and treatment of the contaminated effluent were essential components of a conventional excavation support approach. However, the capacity of the existing water treatment plant was not sufficient to accommodate additional inflow.

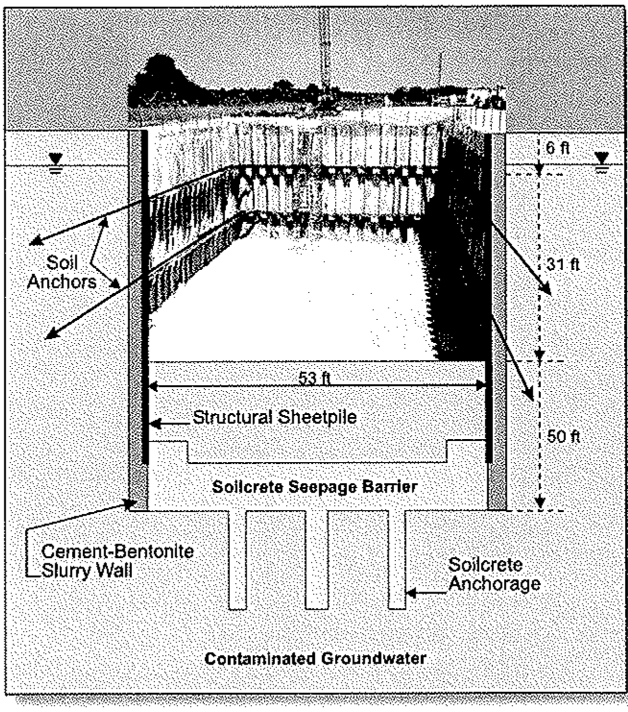
An innovative system of excavation support was therefore developed that would retain the earth pressures as well as provide a hydraulic barrier. This consisted of a perimeter sheetpile wall, placed in a cement-bentonite slurry trench and retained with soil anchors, in conjunction with a jet-grouted horizontal barrier. In essence, a complete cut-off or "bathtub" structure was created prior to any excavation, completely eliminating the need for dewatering and effluent treatment.

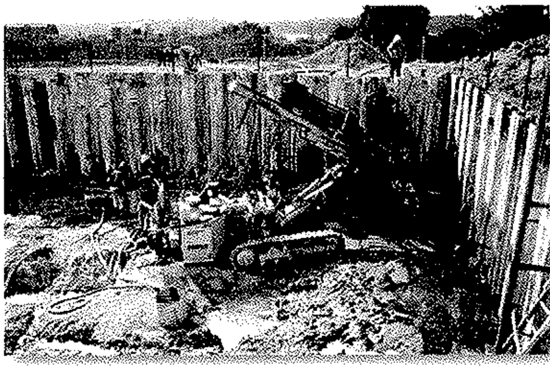
Perimeter Wall

The cement-bentonite slurry trench was constructed in panels to a depth of 46 ft, with the cement-bentonite grout designed to achieve an average unconfined compressive strength of 80 psi in 28 days. The intent of the design was to achieve a strength equal to or greater than the adjacent soil, with a permeability not to exceed 1 x 10^{-5} cm/sec. Interlocking steel sheetpiling was then set to a depth of 40 ft prior to initial set of the slurry. The completed perimeter wall provided a continuous, interlocked structure of very low permeability.

Soilcrete Base

From original grade, a horizontal Soilcrete seepage barrier was constructed at a depth of 53 ft across the entire pit base using triple fluid jet grouting methods. This barrier





Top: The combination of slurry walls, anchored sheetpiling and Soilcrete base provided a watertight structure for excavation of the coal unloading pit.

Above: Soil anchors were installed as pit excavation proceeded.

Owner
AES Cedar Bay
Jacksonville, Florida

Engineer
Ogden Environmental and Energy Services
Oak Ridge, Tennessee

HAYWARD BAKER Geotechnical Construction

KELLER

Project Summary

Cedar Bay, continued...

consisted of overlapping Soilcrete columns designed with a permeability of less than 1 x 10^{-5} cm/sec and an average unconfined compressive strength of 800 psi. Uplift resistance was enhanced by the inclusion of uniformly-spaced, 80 kip capacity, Soilcrete anchors to tie down the slab. Along the edge of the pit, the Soilcrete extended to connect to, and rigidly brace, the toe of the sheetpiling.

As excavation of the pit proceeded, two levels of 142-kip design capacity soil anchors and wales were installed to provide lateral support to the walls.

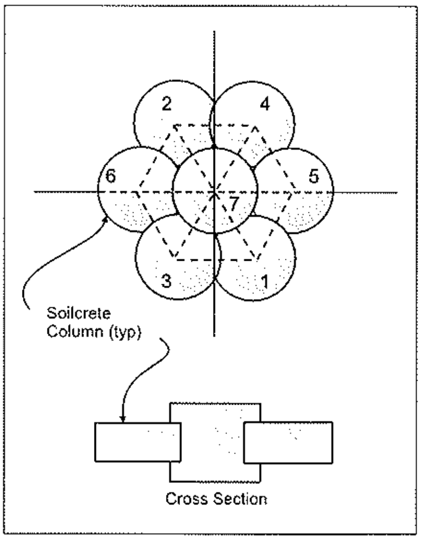
Quality Assurance

Strict quality assurance standards were followed in all phases of the project. During slurry wall construction, the cement-bentonite slurry was sampled daily and cylinders were tested at frequent intervals early in the project to ensure that design assumptions were met.

Prior to jet grouting, a series of test sections were constructed to attest to the ability to construct the design geometry and quality. A specific sequence of work was followed to assure closure of the slab, and layout of the work was checked twice. Daily, in situ Soilcrete sampling and testing furnished data to attest to the strength and consistency of the product.

All 140 anchors were proof-tested to a minimum of 190 kips, and four anchors were performance-tested.

Measured seepage rates into the exposed excavation were less than 5 gpm. This successful and innovative project received the Grand Award for Engineering Excellence from the Consulting Engineers of Tennessee.



Jet Grouting was sequenced to allow individual columns to set up before adjacent columns were installed. Extending the center column above and below surrounding columns ensured a tight connection.

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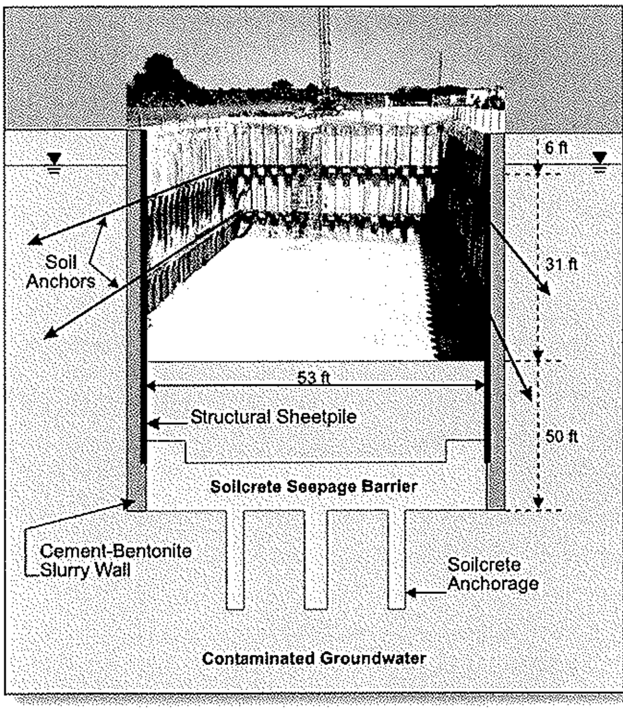

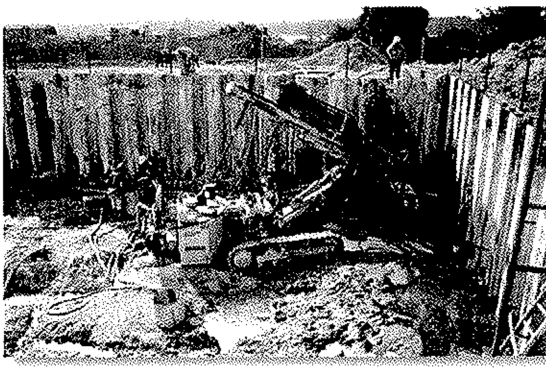

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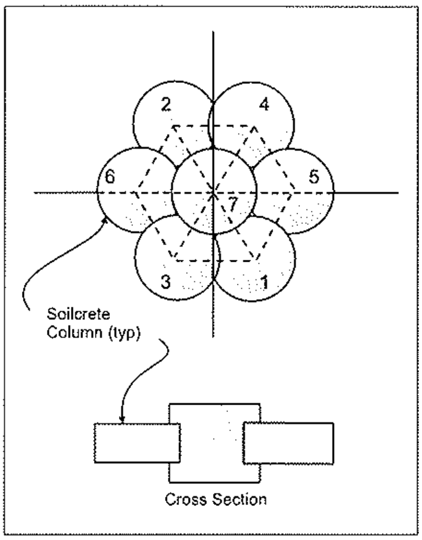

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Integrated Practical Solutions

ACID SULPATE SOIL MANAGEMENT PLAN

STAGE 4 PROJECT, PWCS KOORAGANG COAL TERMINAL

Prepared for
UMWELT (AUSTRALIA) PTY LTD
on behalf of
PORT WARATAH COAL SERVICES LIMITED

Project 49425
OCTOBER 2009

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22 October 2009

ACID SULPHATE SOIL MANAGEMENT PLAN
STAGE 4 PROJECT, KOORAGANG COAL TERMINAL
KOORAGANG ISLAND

1. INTRODUCTION

This Acid Sulphate Soil Management Plan (ASSMP) has been prepared for the proposed construction of a fourth Dump Station and associated conveyor infrastructure at the Port Waratah Coal Services (PWCS) Kooragang Coal Terminal (KCT). The work was carried out for Umwelt on behalf of PWCS.

It is understood that the project includes the following relevant elements:

- A dump station which will be approximately 15 m deep, 12 m wide and 66 m long. The walls will be formed using diaphragm wall or sheet pile wall construction.and the floor will be jet-grouted The excavation will be internally dewatered;
- A conveyor will exit the base of the dump station through a tunnel which will slope up to a surface exit point about 200 m to the east and will be constructed using a similar methodology to the main dump station;
- Various conveyor trestles and associated structures on driven or bored (CFA) piles with shallow pile caps.

The objectives and management strategy proposed are to minimise the potential for adverse environmental impact through appropriate lime treatment/neutralisation and on-site re-use of ASS soils.

This ASSMP is based on the results and recommendations of an assessment of the potential for the development to impact on groundwater (Ref 1) which should be read in conjunction with this ASSMP.

The general ASS management approach is as follows:

- Minimise exposure of ASS (i.e. extent and duration) during excavations/dewatering (i.e. staged works to minimise potential for acid generation);
- Preparation of an appropriate receival and treatment area for the neutralisation of ASS, prior to on-site re-use of treated soils;
- Conduct appropriate lime treatment of ASS to minimise the risk of acid generation and adverse environmental impacts;
- Conduct appropriate testing of baseline conditions (at the treatment site), together with screening tests on untreated and treated ASS to verify that soils have been appropriately treated and are suitable for on-site re-use, and have not resulted in adverse impact at the treatment site;
- Conduct appropriate management, treatment, monitoring and discharge of waters from dewatering activities within ASS;
- Provide appropriate contingency procedures.

This ASSMP has been prepared with reference to the NSW Acid Sulphate Soil Management Advisory Committee (ASSMAC), August 1998 (Ref 2), the Queensland Acid Sulphate Soil Technical Manual, Soil Management Guidelines (QASSIT), November 2002 (Ref 3), and recent experience with similar works in acid sulphate soils.

2. SUMMARY OF ACID SULPHATE SOIL CONDITIONS

Reference 1 indicates that the following acid sulphate soil conditions are present at the site:

- No actual acid sulphate soils present;
- Unit 1 filling does not contain actual or potential acid sulphate soils;

- Unit 2 Clay is potential acid sulphate soils (PASS);
- Unit 3 (Estuarine Aquifer) above and RL -4 NHTG is PASS.

3. POTENTIAL FOR OXIDISING ACID SULPHATE SOILS

The following activities are likely to expose acid sulphate soils to oxidising conditions during construction activities:

- Excavation and internal dewatering for construction of dump station and conveyor;
- Drilling of bored (CFA) piles for support of conveyors and associated structures.

It is expected that Unit 2 Clay soils and Unit 3 Sand soil above an elevation of RL -4 NHTG will require treatment. It is expected that this will comprise between about 7000 m^3 and 10000 m^3 of soil for the dump station / conveyor. Minor quantities may also be disturbed during drilling of piles.

4. MANAGEMENT STRATEGY

4.1 Management of Acid Sulphate Soils

4.1.1 Soil Treatment

Neutralisation of PASS should be undertaken with reference to the ASSMAC and QASSIT guidelines, as discussed below.

All potential acid sulphate soil materials excavated should be transported to a designated treatment area and treated as soon as practicable (i.e. within 12 hrs of excavation).

The ASS treatment area is expected to be on an area containing Unit 1 filling which is permeable and therefore and impermeable liner will be required to protect groundwater. The following procedure is recommended for preparation of the treatment area:

- Grade surface to allow controlled collection of leachate in a catch drain or sump;
- Construct perimeter bunding around the treatment area to a minimum height of 300 mm
- Place impermeable membrane over floor and bunding. For temporary works a HDPE material would be suitable provided that a sand protection layer of at least 0.3 m thickness were placed over it to protect against puncturing;
- Install appropriate erosion and sediment control measures for the perimeter of the treatment area.

Excavated soils once received at the prepared treatment area should be spread out in up to 300 mm layers and treated with an appropriate application of lime (see below).

Suitable neutralising agents for acid sulphate soils include agricultural lime ($CaCO_3$), calcined magnesia (MgO or $Mg(OH)_2$), and dolomite ($MgCO_3.CaCO_3$).

The required dosing rate for lime treatment should be calculated from the following, which includes a factor of safety of 1.5:

Alkali Material Required (kg)

$$\textit{per unit volume of soil } (m^3) = \left(\frac{\%\,S \text{ x } 623.7}{19.98}\right) \text{ x } \frac{100}{\text{ENV (\%)}} \textit{ x D x FOS}$$

Where: *%S = net acidity (% S units);*

623.7 = % S to mol H^+/t;

19.98 = mol H^+/t to kg $CaCO_3$/t;

D = Dry density of soil (t/m^3);

1.5 = safety factor (FOS);

ENV = Effective Neutralising Value (eg. 80% for Grade 1 Agricultural lime).

Note: *The ENV is calculated based on the molecular weight, particle size and purity of the neutralising agent and should be assessed for proposed materials in accordance with QASSIT (Ref 4).*

It is recommended that Grade 1 agricultural lime is used for the neutralisation of potential acid sulphate soils excavated during construction activities.

Stockpiled soil should be limed as soon as practicable following excavation (<12 hrs). If acid sulphate soils cannot be treated within 12 hours of excavation, they should be kept moist to minimise oxidation, prior to treatment with lime.

The neutralising agent and acid sulphate soils should be thoroughly mixed and aerated using, for example, an agricultural lime spreader and excavator, rotary hoe or tillage. The soil should be treated in layers not exceeding 300 mm thick to order to encourage aeration and allow adequate mixing.

The lime rates have been calculated from the limited laboratory results provided in Reference 7, in accordance with the QASSIT and ASSMAC guidelines as follows:

- Unit 2 Clay 30 kg/m^3;
- Unit 3 Sand 6 kg/m^3.

It should be noted that the actual lime rate required will depend on the results of monitoring during neutralisation. Additional lime will be required if monitoring results indicate that appropriate neutralisation has not been achieved. Conversely the liming rate may decrease if monitoring suggests over liming is occurring. Care should be taken so that over-liming does not occur, which could also result in adverse environmental impact.

Sampling and testing (monitoring) should be undertaken in accordance with Section 5.1 to verify the neutralisation treatment. The acceptance criteria is discussed in Section 5.2. Depending on the results of testing, reapplication of lime may be necessary to gain adequate neutralisation.

Upon verification of treatment, the neutralised acid sulphate soils are expected to be suitable for re-use on site.

If off-site disposal of treated ASS is required, the material should be classified in accordance with the NSW DECC waste classification guidelines (Ref 6) prior to disposal at an appropriately licensed landfill. Re-use of the material off-site is prohibited without prior written approval from the NSW DECC, and would require a specific exemption under Reference 6.

4.1.2 Neutralising Leachate

Leachate water collected from the treatment area (i.e. catch drains/sumps), should be neutralised as necessary before disposal. Calcined magnesia (magnesium hydroxide, burnt magnesite, or magnesia) is the recommended neutralising agent as it produces a two-step reaction, which proceeds rapidly at acidic pH and slows down as higher pH is approached, and hence reduces the potential for over shooting to occur.

The amount of neutraliser required to be added to the leachate or discharged groundwater can be calculated from the equation below:

$$\textit{Alkali Material Required (kg)} = \frac{M_{Alkali} \text{ x } 10^{-pH\ initial}}{2 \text{ x } 10^3} \text{ x } V$$

Where: *pH initial = initial pH of leachate*

V = volume of leachate (litres)

M_{Alkali} *= molecular weight of alkali material (g/mole)*

Note: *molecular weight of calcined magnesia* (M_{MgO}) *= 40 g/mole.*

The alkali should be added to the leachate water as a slurry. Mixing of the slurry is best achieved using an agitator.

Notwithstanding regulatory authority requirements, the leachate should meet the water quality criteria presented in Section 5.2 prior to disposal.

4.2 Dewatering

The water extracted from the excavation is proposed to be managed by either of the following:

- Re-injection of the water into the Estuarine Aquifer, with minimal treatment; or
- On-site treatment plant used during construction with disposal either to surface water or by re-injection.

In the instance that direct re-injection is proposed then it is recommended that a flow cell, such as a water tank, be incorporated into the re-injection system to allow monitoring and pH adjustment as part of the re-injection process.

If a dedicated water treatment plant is used then pH adjustment should be incorporated into the treatment process.

Monitoring procedures are provided in Section 5.0 below.

5. MONITORING STRATEGY

5.1 Procedures

5.1.1 Soil Neutralisation/Management

The following inspections and monitoring should be undertaken when excavating acid sulphate soil materials, based on guidelines presented in ASSMAC (Ref 2) and QASSIT (Ref 3):

- Daily inspection of liming operations;
- ASS soil screening tests (i.e. measurements of soil pH in distilled water (pH_F) and pH following oxidation with peroxide (pH_{FOX})), should be undertaken at a frequency of at least one per 100 m^3 or daily, whichever is greater, to verify the neutralisation treatment);
- The ASS screening tests should also be undertaken on untreated materials immediately following excavation to allow comparison of pre and post treatment results to assess the effectiveness of treatment;
- The excavated material should be monitored via screening tests until the neutralisation process has been confirmed to be successful.

The sampling and testing density should be higher (i.e. about one test per 50 m^3) at the commencement of excavation and liming operations or when moving from one management area to the next for the first time, until a suitable lime application rate is determined. The testing frequency could be reduced to one per 200 m^3 if testing indicates consistent and acceptable results.

5.1.2 Dewatering

The pH of extracted water associated with areas of acid sulphate soils should be monitored twice daily (am, pm). Neutralisation should be undertaken if discharge water falls outside the required discharge limits.

As a precautionary measure, initially a higher frequency of testing to that described above is recommended until consistent monitoring results are observed.

5.1.3 Reporting

A record of treatment of acid sulphate soil should be maintained by the contractor and should include the following details:

- Date;
- Chainage/location;
- Time of excavation and backfilling (i.e. time stockpile has been exposed);
- Neutralisation process undertaken;
- Lime rate utilised;
- Results of monitoring (soil and water).

A record of dewatering activities should also include the following:

- Groundwater pH at commencement of dewatering;
- Daily pH monitoring of discharge water

A record should also be maintained confirming contingency measures and additional treatment if undertaken.

5.2 Acceptance Criteria

Water

Notwithstanding regulatory requirements, it is recommended that the ANZECC Guidelines for Fresh and Marine Water Quality, 2000 (Ref 5) be met before discharging any leachate or groundwater to the environment. The recommended criteria for pH is therefore in the range 7.0 to 8.5.

Soil

The recommended criteria for treated soils is as follows:

Table 3 – Recommended Criteria for Treated ASS

Indicator	pH in water (pH_F)	pH following oxidation with peroxide (pH_{Fox})
pH	> 5.0 *	5.0 to 7.7 *

Notes to Table 3:

* Background levels (Ref 1)

Further treatment may be required if monitoring of the treated ASS reveals any of the following properties:

- pH of soil in water (pH_F) is less than the background value (i.e. pH 5.0 - Ref 1), indicating that additional lime treatment is required;
- pH of soil following oxidation with hydrogen peroxide (pH_{FOX}) is outside background values (i.e. pH 5.0 to 7.7 – Ref 1). Additional lime treatment is required if pH_{FOX} is less than 5.0. If pH_{FOX} is greater than 7.7 over-liming has occurred and mixing with untreated ASS is required.

Depending on the results of testing, reapplication of lime or mixing with untreated ASS may be necessary to gain adequate treatment. Care should be taken to ensure over-liming does not occur.

6. CONTINGENCY PLAN

Remedial action will be required if the agreed standards or acceptance criteria are not being achieved. Remedial action shall comprise mixing of additional lime through the excavated material and neutralisation/treatment of groundwater. The required mixing rate to remediate the soil or groundwater should be confirmed via monitoring tests.

If over-liming has occurred, the treated soils should be mixed with untreated ASS to reduce pH levels to within the acceptance criteria. Similarly, waters should be mixed with untreated waters to reduce pH to acceptable levels prior to discharge. Further monitoring of soils and waters should be conducted to confirm adequate treatment has been achieved.

Sufficient storage capacity should be available on-site to allow treatment of groundwater prior to discharge.

During periods of heavy or prolonged rainfall, stockpiles of acid sulphate soils should be appropriately contained/bunded to allow collection of leachate for testing and neutralisation prior to treatment (as required) and disposal (see Section 4.1).

Sufficient lime and flocculants should be stored on site during construction for the neutralisation of acid sulphate soils/reduction in turbidity respectively and contingency measures.

7. LIMITATIONS

Douglas Partners (DP) has prepared this report Umwelt and PWCS for this project at PWCS KCT in accordance with DP's proposal dated 27 July 2009 and acceptance received from Umwelt dated 28 July 2009. The work was carried out under DPs Conditions of Engagement in tandem with Umwelt Subconsultant Conditions of Engagement as amended by DP on 29 July 2009. This report is provided for the exclusive use of the Umwelt and PWCS for the specific project and purpose as described in the report. It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party.

The results provided in the report are considered to be indicative of the sub-surface conditions on the site only to the depths investigated at the specific sampling and/or testing locations, and only at the time the work was carried out. DP's advice may be based on observations, measurements, tests or derived interpretations. The accuracy of the advice provided by DP in this report is limited by unobserved features and variations in ground conditions across the site in areas between test locations and beyond the site boundaries or by variations with time. The advice may be limited by restrictions in the sampling and testing which was able to be carried out, as well as by the amount of data that could be collected given the project and site constraints. Actual ground conditions and materials behaviour observed or inferred at the test locations may differ from those which may be encountered elsewhere on the site. Should variations in subsurface conditions be encountered, then additional advice should be sought from DP and, if required, amendments made.

This report must be read in conjunction with the attached "Notes Relating to This Report" and any other attached explanatory notes and should be kept in its entirety without separation of individual pages or sections. DP cannot be held responsible for interpretations or conclusions from review by others of this report or test data, which are not otherwise supported by an expressed statement, interpretation, outcome or conclusion stated in this report. In preparing this report DP has necessarily relied upon information provided by the client and/or their agents.

DOUGLAS PARTNERS PTY LTD

Reviewed by:

Will Wright
Principal

Stephen Jones
Principal

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6. NSW DECC, "Waste Classification Guidelines, Part 1 – Classifying Waste", April 2008.

7. Douglas Partners, "Report on Acid Sulphate Soil Management Plan, Dump Station and Conveyor Tunnel, PWCS Stage 3 Expansion, Kooragang Coal Loader", Project 3100C15, February 2000.

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NOTES RELATING TO THIS REPORT

Introduction

These notes have been provided to amplify the geotechnical report in regard to classification methods, specialist field procedures and certain matters relating to the Discussion and Comments section. Not all, of course, are necessarily relevant to all reports.

Geotechnical reports are based on information gained from limited subsurface test boring and sampling, supplemented by knowledge of local geology and experience. For this reason, they must be regarded as interpretive rather than factual documents, limited to some extent by the scope of information on which they rely.

Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726, Geotechnical Site Investigations Code. In general, descriptions cover the following properties - strength or density, colour, structure, soil or rock type and inclusions.

Soil types are described according to the predominating particle size, qualified by the grading of other particles present (eg. sandy clay) on the following bases:

Soil Classification	Particle Size
Clay	less than 0.002 mm
Silt	0.002 to 0.06 mm
Sand	0.06 to 2.00 mm
Gravel	2.00 to 60.00 mm

Cohesive soils are classified on the basis of strength either by laboratory testing or engineering examination. The strength terms are defined as follows.

Classification	Undrained Shear Strength kPa
Very soft	less than 12
Soft	12—25
Firm	25—50
Stiff	50—100
Very stiff	100—200
Hard	Greater than 200

Non-cohesive soils are classified on the basis of relative density, generally from the results of standard penetration tests (SPT) or Dutch cone penetrometer tests (CPT) as below:

Relative Density	SPT "N" Value (blows/300 mm)	CPT Cone Value (q_c — MPa)
Very loose	less than 5	less than 2
Loose	5—10	2—5
Medium dense	10—30	5—15
Dense	30—50	15—25
Very dense	greater than 50	greater than 25

Rock types are classified by their geological names. Where relevant, further information regarding rock classification is given on the following sheet.

Sampling

Sampling is carried out during drilling to allow engineering examination (and laboratory testing where required) of the soil or rock.

Disturbed samples taken during drilling provide information on colour, type, inclusions and, depending upon the degree of disturbance, some information on strength and structure.

Undisturbed samples are taken by pushing a thin-walled sample tube into the soil and withdrawing with a sample of the soil in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Details of the type and method of sampling are given in the report.

Drilling Methods.

The following is a brief summary of drilling methods currently adopted by the Company and some comments on their use and application.

Test Pits — these are excavated with a backhoe or a tracked excavator, allowing close examination of the in-situ soils if it is safe to descent into the pit. The depth of penetration is limited to about 3 m for a backhoe and up to 6 m for an excavator. A potential disadvantage is the disturbance caused by the excavation.

Large Diameter Auger (eg. Pengo) — the hole is advanced by a rotating plate or short spiral auger, generally 300 mm or larger in diameter. The cuttings are returned to the surface at intervals (generally of not more than 0.5 m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube sampling.

Continuous Sample Drilling — the hole is advanced by pushing a 100 mm diameter socket into the ground and withdrawing it at intervals to extrude the sample. This is the most reliable method of drilling in soils, since moisture content is unchanged and soil structure, strength, etc. is only marginally affected.

Continuous Spiral Flight Augers — the hole is advanced using 90—115 mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or in-situ testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are very disturbed and may be contaminated. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively lower reliability, due to remoulding, contamination or softening of samples by ground water.

Non-core Rotary Drilling — the hole is advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from 'feel' and rate of penetration.

Rotary Mud Drilling — similar to rotary drilling, but using drilling mud as a circulating fluid. The mud tends to mask the cuttings and reliable identification is again only possible from separate intact sampling (eg. from SPT).

Continuous Core Drilling — a continuous core sample is obtained using a diamond-tipped core barrel, usually 50 mm internal diameter. Provided full core recovery is achieved (which is not always possible in very weak rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation.

Standard Penetration Tests

Standard penetration tests (abbreviated as SPT) are used mainly in non-cohesive soils, but occasionally also in cohesive soils as a means of determining density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, "Methods of Testing Soils for Engineering Purposes" — Test 6.3.1.

The test is carried out in a borehole by driving a 50 mm diameter split sample tube under the impact of a 63 kg hammer with a free fall of 760 mm. It is normal for the tube to be driven in three successive 150 mm increments and the 'N' value is taken as the number of blows for the last 300 mm. In dense sands, very hard clays or weak rock, the full 450 mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form.

- In the case where full penetration is obtained with successive blow counts for each 150 mm of say 4, 6 and 7

  as 4, 6, 7
  N = 13

- In the case where the test is discontinued short of full penetration, say after 15 blows for the first 150 mm and 30 blows for the next 40 mm

  as 15, 30/40 mm.

The results of the tests can be related empirically to the engineering properties of the soil.

Occasionally, the test method is used to obtain samples in 50 mm diameter thin walled sample tubes in clays. In such circumstances, the test results are shown on the borelogs in brackets.

Cone Penetrometer Testing and Interpretation

Cone penetrometer testing (sometimes referred to as Dutch cone — abbreviated as CPT) described in this report has been carried out using an electrical friction cone penetrometer. The test is described in Australian Standard 1289, Test 6.4.1.

In the tests, a 35 mm diameter rod with a cone-tipped end is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the friction resistance on a separate 130 mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are connected by electrical wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck.

As penetration occurs (at a rate of approximately 20 mm per second) the information is plotted on a computer screen and at the end of the test is stored on the computer for later plotting of the results.

The information provided on the plotted results comprises: —

- Cone resistance — the actual end bearing force divided by the cross sectional area of the cone — expressed in MPa.
- Sleeve friction — the frictional force on the sleeve divided by the surface area — expressed in kPa.
- Friction ratio — the ratio of sleeve friction to cone resistance, expressed in percent.

There are two scales available for measurement of cone resistance. The lower scale (0—5 MPa) is used in very soft soils where increased sensitivity is required and is shown in the graphs as a dotted line. The main scale (0—50 MPa) is less sensitive and is shown as a full line.

The ratios of the sleeve friction to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1%—2% are commonly encountered in sands and very soft clays rising to 4%—10% in stiff clays.

In sands, the relationship between cone resistance and SPT value is commonly in the range:—

$$q_c \text{ (MPa)} = (0.4 \text{ to } 0.6) \text{ N (blows per 300 mm)}$$

In clays, the relationship between undrained shear strength and cone resistance is commonly in the range:—

$$q_c = (12 \text{ to } 18)\, c_u$$

Interpretation of CPT values can also be made to allow estimation of modulus or compressibility values to allow calculation of foundation settlements.

Inferred stratification as shown on the attached reports is assessed from the cone and friction traces and from experience and information from nearby boreholes, etc. This information is presented for general guidance, but must be regarded as being to some extent interpretive. The test method provides a continuous profile of engineering properties, and where precise information on soil classification is required, direct drilling and sampling may be preferable.

Hand Penetrometers

Hand penetrometer tests are carried out by driving a rod into the ground with a falling weight hammer and measuring the blows for successive 150 mm increments of penetration. Normally, there is a depth limitation of 1.2 m but this may be extended in certain conditions by the use of extension rods.

Two relatively similar tests are used.

- Perth sand penetrometer — a 16 mm diameter flat-ended rod is driven with a 9 kg hammer, dropping 600 mm (AS 1289, Test 6.3.3). This test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.
- Cone penetrometer (sometimes known as the Scala Penetrometer) — a 16 mm rod with a 20 mm diameter cone end is driven with a 9 kg hammer dropping 510 mm (AS 1289, Test 6.3.2). The test was developed initially for pavement subgrade investigations, and published correlations of the test results with California bearing ratio have been published by various Road Authorities.

Laboratory Testing

Laboratory testing is carried out in accordance with Australian Standard 1289 "Methods of Testing Soil for Engineering Purposes". Details of the test procedure used are given on the individual report forms.

Bore Logs

The bore logs presented herein are an engineering and/or geological interpretation of the subsurface conditions, and their reliability will depend to some extent on frequency of sampling and the method of drilling. Ideally, continuous undisturbed sampling or core drilling will provide the most reliable assessment, but this is not always practicable, or possible to justify on economic grounds. In any case, the boreholes represent only a very small sample of the total subsurface profile.

Interpretation of the information and its application to design and construction should therefore take into account the spacing of boreholes, the frequency of sampling and the possibility of other than 'straight line' variations between the boreholes.

Ground Water

Where ground water levels are measured in boreholes, there are several potential problems;

- In low permeability soils, ground water although present, may enter the hole slowly or perhaps not at all during the time it is left open.
- A localised perched water table may lead to an erroneous indication of the true water table.
- Water table levels will vary from time to time with seasons or recent weather changes. They may not be the same at the time of construction as are indicated in the report.
- The use of water or mud as a drilling fluid will mask any ground water inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water observations are to be made.

More reliable measurements can be made by installing standpipes which are read at intervals over several days, or perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from a perched water table.

Engineering Reports

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg. a three storey building), the information and interpretation may not be relevant if the design proposal is changed (eg. to a twenty storey building). If this happens, the Company will be pleased to review the report and the sufficiency of the investigation work.

Every care is taken with the report as it relates to interpretation of subsurface condition, discussion of geotechnical aspects and recommendations or suggestions for design and construction. However, the Company cannot always anticipate or assume responsibility for:

- unexpected variations in ground conditions — the potential for this will depend partly on bore spacing and sampling frequency
- changes in policy or interpretation of policy by statutory authorities
- the actions of contractors responding to commercial pressures.

If these occur, the Company will be pleased to assist with investigation or advice to resolve the matter.

Site Anomalies

In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some later stage, well after the event.

Reproduction of Information for Contractual Purposes

Attention is drawn to the document "Guidelines for the Provision of Geotechnical Information in Tender Documents", published by the Institution of Engineers, Australia. Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. The Company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Site Inspection

The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.