PROGRAM'S SECOND SET OF ANSWERS TO EAP REPORT

On Nov. 4th 2010, the EAP provided the Program with a new set of comments with respect to the Program's first set of answers regarding the EAP report. Please find hereunder the Program's second set of answers.

EAP question 1: Design flow

The flow that was presented during the Workshop, was based on the population of 229,800 while the updated design stated that the flowrate was based on the population of 250,000. However, the flowrate shown in Page 5 is the same as presented in Workshop. What was the reason that the flowrate was unchanged even though the design population was increased?

Program's answer 1

Design flow presented during the workshop was based on different per capita flows for the two population components, i.e., existing population and future population. Existing population flow of 280.50 liters/capita/day (lcd) was based on historical data while future population flow was based on a domestic flow of 220 lcd with an additional 108 lcd to account for inflow/infiltration, industrial and commercial contribution. Consequently total per capita flow from future population was 328 lcd, about 16.7% higher than historical flows.

Workshop design dry weather flows:

Existing population = (170,613 PE * 280.50 lcd) = 47.86 MLD
Future population = (65,106 PE * 328 lcd) = 21.35 MLD
Total Dry Weather Design Flow (235,718 PE) = 69.21 MLD
Annual Average Flow = (1.264 PF * 69.21) = 87.48 MLD

The updated design flow for a population of 250,000 was based on a uniform per capita flow for both the existing population and future population per discussions and conclusions during the workshop. Updated population figures, provided by the City just before the workshop, resulted in a small decrease in the historical per capita flows from 280.50 lcd to 276.95 lcd.

Updated design dry weather flows:

Dry Weather Design Flow = (250,000 PE * 276.95 lcd) = 69.24 MLD
Annual Average Flow = (1.264 PF * 69.24) = 87.52 MLD

In both cases the Annual average design flow was rounded to 87.50 MLD. Thus the flow remains the same even though the assumptions changed.

EAP question 2: Design population

What will be the population for the final design and for cost analysis: 250,000 or 270,000?

Program's answer 2

The population for the final design and cost analysis is 250 000 PE as it has been agreed during the workshop in September 2010.

EAP question 3: Loads and concentrations

During the Workshop, the Program and EAP agreed not to use per-capita loading from MOE data. Hence, EAP presumes that the loading in the updated design (Table 2) reflects it. Could the Program explain how Table 2 was prepared? Average differences between Workshop design and the updated design were TSS (9% down), BOD (5% up), TN (3% up) and TP (5% down).

Program's answer 3

The workshop load and concentration projections followed a similar procedure outlined in Response 1 above, i.e., historical per capita values for existing population and MOE data for future population. As noted in the EAP comment, the updated load and concentration projections are based on historical per capita values for the entire future population of 250,000. As noted above in Response 1, a change in the historical population figures resulted in a small drop in per capita values. Tables below show the calculations used in the two projections.

Load Calculations - Annual Average Day (WORKSHOP)

		TSS	BOD	TKN	TP	
Existing 2005-2010 Dry Weather Population	170,613					
Dry Weather Per Capita Load (Kg/cap/day)		0.0585	0.0753	0.0140	0.0022	(From actual 2005 - 2010 April Data)
Year 2031 Existing Population Dry Weather Load (Kgs/day)		9,980	12,854	2,391.0	372.7	
Existing 2005-2010 Dry Weather Loads		9,980.3	12,854.0	2,391.0	372.7	
Existing 2005-2010 Annual Average Loads		11,383.6	13,025.7	2,460.8	384.4	
Dry Weather to Annual Average Peaking Factor		1.14	1.01	1.03	1.03	
Year 2031 Existing Population Annual Average Load (Kgs/day)		11,384	13,026	2,461	384	
Future 2031 Population	65,106					
Annual Average Per Capita Load (Kg/cap/day)		0.095	0.075	0.015	0.003	(From Ontario MOE Guidelines - Stantec PDR)
Year 2031 Future Population Annual Average Load (Kgs/day)		6,185	4,883	976.6	195.3	
SEWPCC Population - Year 2031	235,718	Includes a 10 Percent factor in 2031 population increase				
SEWPCC Annual Average Load - Year 2031 (Kg/Cap/day)		17,569	17,909	3,437	580	
SEWPCC Annual Average Flow - Year 2031 (MLD)		87.50	87.50	87.50	87.50	
Parameter Concentration (mg/L)		201	205	39.3	6.6	
Windsor Park Flow - Year 2031 (MLD)		-	-	-	-	
Windsor Park Load - Year 2031 (Kgs/day)		-	-	-	-	(Included in 2031 Population)
SEWPCC Total Load - Year 2031 (Kgs/day)		17,569	17,909	3,437	580	
SEWPCC Total Flow - Year 2031 (MLD)		87.50	87.50	87.50	87.50	
Parameter Concentration (mg/L)		201	205	39.3	6.6	

Load Calculations - Annual Average Day (UPDATED after WORKSHOP)

		TSS	BOD	TKN	TP	
Existing 2005-2010 Dry Weather Population	181,202					
Dry Weather Per Capita Load (Kg/cap/day)		0.0578	0.0744	0.0138	0.0022	(From actual 2005 - 2010 April Data)
Year 2031 Existing Population Dry Weather Load (Kgs/day)		10,466	13,479	2,507.3	390.9	
Existing 2005-2010 Dry Weather Loads		9,980.3	12,854.0	2,391.0	372.7	
Existing 2005-2010 Annual Average Loads		11,383.6	13,025.7	2,460.8	384.4	
Dry Weather to Annual Average Peaking Factor		1.14	1.01	1.03	1.03	
Year 2031 Existing Population Annual Average Load (Kgs/day)		11,937	13,659	2,580	403	
Future 2031 Population	68,798					
Annual Average Per Capita Load (Kg/cap/day)		0.0578	0.0744	0.0138	0.0022	
Year 2031 Future Population Annual Average Load (Kgs/day)		3,974	5,118	951.9	148.4	
SEWPCC Population - Year 2031	250,000					
SEWPCC Annual Average Load - Year 2031 (Kg/Cap/day)		15,910	18,776	3,532	551	
SEWPCC Annual Average Flow - Year 2031 (MLD)		87.50	87.50	87.50	87.50	
Parameter Concentration (mg/L)		182	215	40.4	6.3	
Windsor Park Flow - Year 2031 (MLD)		-	-	-	-	
Windsor Park Load - Year 2031 (Kgs/day)		-	-	-	-	(Included in 2031 Population)
SEWPCC Total Load - Year 2031 (Kgs/day)		15,910	18,776	3,532	551	
SEWPCC Total Flow - Year 2031 (MLD)		87.50	87.50	87.50	87.50	
Parameter Concentration (mg/L)		182	215	40.4	6.3	

EAP question 4: Reactor tank sizing

BNR tank size in the updated design increased especially in Option #2 and #3. EAP also noted that the flow to BNR system was reduced to 120 ML (update) from 125ML (Workshop).

1) Aerobic tank was increased from 16500 ML to 18500 ML (12%) in the updated Option #2. What was the reason of the increase of aerobic zone in Option #2? Is it due to the increase of BOD (5%) and TN (3%)?

Program's answer 4.1

The increase in COD, BOD_5 and TN load is only a part of the answer for the increase in aerobic tank volume. The main reason for this increase is the discussion during the workshop with respect to the possibility to shut down one flow train for maintenance (redundancy strategy). We stated that this was possible but all the media had to be transferred into the remaining three operating flow trains to maintain nitrification. Consequently, we designed the aerobic tank to reach the maximum media filling (around 67%) when one line is shut down. Thus in normal conditions, the media filling of the tanks is around 48% which is close the theoretical value of 50% media filling for good performances of IFAS. This is the best compromise between design and operational flexibility.

2) Workshop stated that further optimization will be done in terms of the anaerobic tank sizing. Workshop presented the difference between Simulo and Biowin and the Program acknowledged that the Simulo model will not be used. However, the Program neglected the BIOWIN simulation, which is proven in terms of biological P removal in North America, because of its conservativeness. As a result, the updated design does not contain the optimization of anaerobic tank size even though the TP loading in the updated design was 5% less than the earlier Workshop design.

Program's answer 4.2

During the workshop we agreed to use the results of Biowin in Simulo. Consequently in the updated concept papers, the Simulo parameters have been adapted to obtain the same results as Biowin with a margin of safety. In the updated versions of options 2 and 3 the chemical use has been reduced or completely eliminated to obtain more than 80% efficiency removal only with biological P.

The retention time for the anaerobic zone is kept at 1.5 hour. This could be reduced to 1 hour, but in this case this would lead to the increase of the average concentration in the all activated sludge trains from 4.0 g MLSS /l to 4.2 g MLSS/l which is not acceptable. Consequently, to remain at 4.0 g MLSS/l, we would have to increase the aerobic part and the total volume for activated sludge would then remain the same than with a 1.5 hour retention time.

3) EAP believes that Option #2 and #3 can be further optimized just as Option #4 was.

Program's answer 4.3

Options 2, 3 and 4 have been optimized equally at a sufficient level for a preliminary design and a process selection purpose. When the process option is finally selected, the detailed design will allow additional level of optimization which may impact total reactor volumes and also in regards with operation, maintenance issues, hydraulic constraints, etc… Moreover a process optimization cannot be assessed solely as it is tightly linked to the process guarantee provided by the contractor and its cost.

EAP question 5

EAP question 5 : Expected TN quality in the final effluent (based on tables from each option)

The updated design presented the final effluent TN quality of 12 mg/L (Option #2, Table 26), 12 mg/L (Option #3, Table 59) and 14 mg/L (Option #4, Table 75).

Why do the options have different design TN limits? Does this mean that Option #2 and #3 can be further optimized to bring them in-line with Option #4?

<u>Program's answer 5</u>
There is a mistake in Table 75 for option 4. The TN value should be 12 mg/L for "design conditions" as for the options 2 and 3.
All the options have been designed to obtain an overall 14 mg/l TN at the outlet all the year round. The difference between this 14 mg/l and the regulatory 15 mg/l is just a safety factor which is the same for all 3 options. Anyway, to respect the 14 mg/l TN at the outlet, we have to take into consideration the blending effect between the biological treatment stream and the CSO stream during the spring max month. That's the reason why we have considered 12 mg/l TN as a design condition.

Inquiries to the "Answers to EAP report"

EAP question 6 : Comment on P3
Why does the design intend to use the external carbon source in pre-anoxic zone? EAP have never experienced a design with external carbon to a pre-anoxic zone to, presumably, reduce the redox potential. The idea is to provide more anoxic retention of sludge – rather than do it in the final clarifier - and to achieve endogenous decrease of ORP. Our concern is with an increase of complexity and costs here.

<u>Program's answer 6</u>
The Program's answer previously provided used the term "external source" mistakenly. The "external source of C" noted by the Program in its previous answer refers to the primary effluent which contains C as noted at the beginning of that response.
We agree that the aim of this zone is to provide more anoxic retention time to achieve endogenous denitrification and to reduce ORP. We do not intend to add a real external source of C but we intend to put a part of the inlet flow in this zone to help the denitrification in the next anoxic zone.
There are no additional costs or complexity in the proposed system using primary effluent C in the anoxic zone and can be optimised even during start up or operation.
This is an optimisation issue that will be treated when the final option is retained.

EAP question 7 : Comments on P5, P6 and P7
The Program's answers seem to agree with EAP's comments, but the updated design does not reflect this agreement. The clarifier sizing in the updated Option #2 and #3 was unchanged – in other words it was left over-sized.

<u>Program's answer 7</u>
These questions and answers referred to Option 1 that was designed with SVI = 150 ml/g, velocity = 0.6 m/hr and sludge solids rate = 4.5 kg/m2/hr with chemical dosing. This is not the design of options 2 and 3. It is important to remember that chemical dosing is not a 24/7/365 operation. There are times in the year when no chemical may be added.

<u>Design of clarifiers for option 2:</u>

IFAS after primary settlers gives a SVI of around 130-140 ml/g
Velocity in the clarifiers is considered = 0.75 m/hr at maximum conditions (that can last 24 hours)
Sludge solids rate = 6 kg/m2/hr
Moreover, if we consider that we don't have any chemical in the AS, these values are really not conservative at all.

<u>Design for clarifiers for Option 3</u>
Average load AS after a primary settler gives a SVI of around 140-150 ml/g
Velocity in the clarifiers is considered = 1.2 m/hr at maximum conditions (that can last 24 hours)
Sludge solids rate = 7.5 kg/m2/hr
Moreover, if we consider that we don't have any chemical in the AS, these values are really not conservative at all.

Design of clarifiers cannot be optimized more than this with respect to process guarantee without exposing the Program to some technical.

EAP question 8 : Comment on P8
The explanation is clear but do we really believe that the influent TSS max day was 133,793 kg/d versus the other max days ~69,000 kg/d? This may be so, but seems very extreme.
Also, there is a difference in design if I receive 133,793 kg/d on one day and the remaining days are in the order of 20,000 kg/d.

<u>Program's answer 8</u>
The Program agrees with the EAP that the projected peak day Spring TSS load of 133,793 Kg/day is an extreme case. The data point that resulted in this extremely high peak day projection refers to the May 13th 2009 data set. The EAP is again correct that the remaining days are in the order of 20,000 Kg/day. Actual data indicates that the TSS load over the following five days ranged from 11,315 – 17,150 Kg/day while on the previous day it was around 23,000 Kg/day.

Historical data analysis was discussed in Part II - Section I SEWAGE CHARACTERIZATION of the SEWPCC Process Selection Report (PSR) submitted to the EAP prior to the Workshop. As discussed in Subsection 4.1 Existing Wastewater Flows and Loads, significant data evaluation was conducted to determine outliers. Probability plots were used to verify if the extreme values were actual outliers. In addition, the day in question was from a data set that had been validated by the City for sampling and analytical inaccuracies. From the analysis, it was noted that high flows of 172 MLD (2.84 times the 5 year average) resulted in washout of settled solids on May 13th 2009. The resulting influent TSS concentration was 504 mg/L while that the next day was 98 mg/L at an influent flow of 175 MLD.

To further verify if eliminating these would result in a significant decrease, an analysis after eliminating data falling beyond 3 standard deviations was conducted as discussed in Subsection 4.1 of the PSR. As noted in Table 3 of the PSR, a significant difference in either concentration or mass load was not noted between the two data sets.

As such events do occur in collection systems and there was no specific reason to treat them as outliers, the Program decided to retain them.

Anyway, the maximum day does not enter in any of the design calculations. The design has been based on the maximum month load (the spring max month). The design has then been verified for the maximum week and when necessary for the maximum day. This verification only aims to check what would be necessary in operation to forecast the event and to minimise the consequences on the performances. Thus this issue will be of extreme importance for start-up and operation but doesn't currently impact the design.

EAP question 9 : Comment on P9

The reminder is appreciated, however, the issue is that for South End, there are primary settling tanks already constructed and operated. While if one were starting from scratch without existing equipment, the advantages noted are true when attempting to achieve the same % TSS removal, this application is not. The issue is why construct high rate primary treatment units when using CEPT at lower chemical dosing rates than a high rate primary treatment unit and no microsand can achieve approximately 75% TSS removal versus 85% removal. What is the benefit of the additional 10% TSS removal?

Program's answer 9

The reasons for which the Program proposes a high rate primary treatment unit instead of a CEPT are both technical and financial.

Technical reasons:

- Existing primary settlers are not sufficient to treat all the flows using the CEPT process. Consequently, a new primary settlers train to be used as a CEPT would have to be added to keep a velocity to less than 5 m/h to insure a good removal efficiency without adding prohibitive doses of chemicals. Based on Stantec's calculation (CDR) a total CEPT area of about 2900 m2 would be needed.
- The chemical dosing in a CEPT is less optimised than in a high rate primary treatment unit because of the microsand and the efficient high mixing intensity coagulation zones. Thus it is acknowledged that the doses are usually at least 25% higher in CEPT.
- A rough calculation showed that a CEPT at a velocity around 5 m/h with 100 mg/l of $FeCl_3$ as pure product can reach around 55% BOD_5 removal and 75% TSS removal, while a high rate primary treatment unit with 75 mg/l of $FeCl_3$ as pure product can reach around 65% BOD_5 removal and 85% TSS removal.
- The design of the existing primary settlers is not adapted to store large quantity of chemical sludge. Their depth is 4.3 m, so there may be a risk of sludge rising to the surface if the flow is high (flow limit to be assessed but assumed > 120 MLD).
- All the existing sludge pumps and piping would need to be replaced with bigger capacities.
- The most difficult technical point is to reach the necessary BOD_5 removal efficiency. The regulation limits for BOD_5 is 25 mg/l as 30 day rolling average. This might be very difficult to reach when mixing the biological effluent and wet weather streams

during spring and summer, if the efficiency on the chemical treatment is not well optimized (very difficult to control).
- It is also important to remember that current operation includes addition of WAS to the primary settlers to maintain a thinner primary sludge concentration to assist in sludge withdrawal. SEWPCC has had significant issues when the primary sludge is thick. Proposed design does not include co-settling as currently practiced. Adding chemicals for CEPT will result in a thicker primary sludge and increase sludge withdrawal problems requiring significant modifications to the sludge mechanisms.

Economic reasons:
- Less civil works in high rate primary treatment unit than in CEPT as it is an intensive process (bigger CEPT should be built to reach the results).
- Less chemicals (from a dosing point of view and also from a daily load) in a high rate primary treatment unit to reach better results than CEPT.
- The chemical dosing in a CEPT solution would be done on the total flow, while with a high rate primary treatment unit, the chemical would apply only on the exceeding part of the flow (175 MLD instead of 300 MLD). The savings can thus be very significant.

EAP question 10 : Comment on P10
So does CEPT not meet the effluent quality required? This needs to be clearly stated and not implied.

Program's answer 10
Please refer to the previous answer.

EAP question 11 : Comment on P27
During the Workshop, it was also stated that Simulo is not able to properly design the biological P removal. Still, the Program uses Simulo exclusively for the updated design. Why?

Program's answer 11
As mentioned above and accepted during the workshop, the results of Simulo have been adapted to obtain the same results as Biowin for bio P removal with a margin of safety.

EAP question 12 : Comment on Phosphorus recovery (1.5)
1) Use of chemicals for struvite recovery
Struvite recovery requires an additional process, but it would be the best available method to recover phosphorus and it is only achievable with sludge from Option #2 and #3 as they do not add ferric. Struvite recovery requires chemical, so it will affect the operation cost. However, the chemical will be used for the recovery of phosphorus, not to bind phosphorus to an unrecoverable form.
EAP understands the economy of scale in P recovery. P-recovery appears economically feasible at SEWPCC but could also be done at NEWPCC where it could be more profitable when done from combined sludge at the NEWPCC site. As the

protection of the digestion and gas system from sulphides at the NEWPCC currently requires ferric, the eventual P recovery there would have to be done before the methanogenic digestion.

2) Amount of recoverable P in Option #4
We agreed with the idea that Option #4 still has some portion of biologically recoverable P from BAF units. However, the amount shown in Answer should be re-visited. Here's a rough estimation based on the yearly average from updated design of Option #4.
P in raw influent: 6.3 mg/L and 552 kg P/d
P in primary effluent: 3.0 mg/L and 264 kg P/d
P in final effluent: 1.0 mg/L and 87.5 kg P/d

The amount of P in primary sludge will be 228 kg P/d (552-264). The amount of primary sludge in design is 14,123 kg TSS/d. So, %P will be 2.04% (228/14123), looks reasonable as chemicals added.

The amount of P in secondary sludge will be 177 kg P/d (264-87.5). The amount of secondary sludge in design is 7,730 kg TSS/d. So, the %P will be 2.28% (177/7730). It seems that %P in secondary sludge is too large, considering that cell growth is the only P removal mechanism in the updated Option #4. Therefore, the amount of recoverable P from Option #4 should be checked.

Program's answer 12.1
The amount of P in the secondary sludge is confirmed. The slightly higher P content of the sludge (per M&E typical P in bacterial cell is 2%) can be explained by two different factors. First, the P removal mechanism of the BAF is through biological growth, but also TSS removal (filtration), so that some particulate P is removed through the filtration process, while the soluble part is removed by biological growth. Second, the backwash water coming out of the BAF do contain particulate P (from the filtration process and from biological growth), but also some soluble P (from the backwash water, which contain about 1/4th of BAF influent and 3/4th of BAF effluent). This soluble fraction is then precipitated (using coagulant) into the backwash clarification system, and thus becomes particulate as well. The total is thus composed of these three particulate P sources (filtered out solids, biological growth, and BW clarification P precipitation), which explains the relatively high TP concentration of the BAF sludge.

In summary, the phosphorus recovery can take place at SEWPCC but it can also be planned for the NEWPCC site, with appropriate sludge handling. The Program should explicitly state that in design and submissions to the regulator and include the differences in recoverable phosphorus from the different Options in the weighing/scoring.

Program's answer 12.2
The Program agrees that P recovery should be stated in submissions to the Regulator. Phosphorus could be recovered to a different extent for each option, based on the SEWPCC process selection and also on the global sludge management approach. P recovery options are for the moment based on the same

principle, which is struvite precipitation. This struvite could be produce following two approaches:

1. Phostrip-type process: struvite formation on a concentrated phosphorus stream, produced by a P release process out of a biological phosphorus removal system. This is then applicable to options 2 and 3. Under this approach, P could be released from the WAS by adding an external carbon source to the WAS stream. This carbon source could be a pure commercial VFA product bought from a supplier (as an example acetate), or site-generated VFAs produced out of primary sludge fermentation. This VFA would then be dosed to the WAS resulting in a P release by the bioP organisms. This P-rich stream would then be sent to a struvite precipitation reactor, where all required chemicals (mainly magnesium, but also ammonia and pH control chemicals) would be added to allow struvite formation. The struvite could then be removed from the process in a solid form to be used as fertilizer. The remaining sludge (P depleted WAS and primary sludge) would need to be further processed and/or disposed of just like typical wastewater treatment sludge. The quantities of P that could be recovered under this approach are presented in the following table.

Approach 1: Phostrip-type struvite precipitation process			
	Option 2 and 3		**Option 4**
	mg/L	kg/d	kg/d
P inlet	6.3	554.4	554.4
P primary effluent	5.9	519.2	259.6
P secondary effluent	1	88	88
Secondary treatment P removal	4.9	431.2	171.6
particulate P (from influent) removed through clarification	0.3		
P removed by cell synthesis	1.2	105.6	
BioP removal	3.4	299.2	0
Phostrip-type struvite recovery potential		**299.2**	**0**
Phostrip-type struvite recovery potential as compared to total P removed from the wastewater		**64%**	**0**

The table shows that out of the 554 kg/d of P fed to the SEWPCC, 88 kg/d leave with the effluent, so that 466 kg/d are removed by the WWTP. Out of these 466 kg/d, 299 kg/d would be available for struvite precipitation, which represent 64% of the P removed by the WWTP. This approach is not suitable for option 4.

2. Struvite precipitation downstream of anaerobic sludge digestion process. Under this approach, a phosphorus-rich steam would be produced out of the anaerobic digestion process, following sludge thickening and/or dewatering. This P-rich stream would also be rich in ammonia, so that only magnesium and possibly some pH control chemicals would be required to enhance struvite formation. In the anaerobic digestion process, all the P which is not chemically bounded to a metal salt (meaning all non-coagulated P) could become soluble and thus available for struvite production. This includes raw sewage particulate P, as well as all P associated with microbial growth. The following table summarizes the struvite precipitation potential of the three options downstream anaerobic digestion.

Approach 2: Struvite precipitation downstream anaerobic digestion		
	Option 2 and 3	**Option 4**

	mg/L	kg/d	mg/L	kg/d
P inlet	**6.3**	**554.4**	**6.3**	**554.4**
Particulate P	1.26	110.9	1.26	110.9
soluble P	5.04	443.5	5.04	443.5
P primary effluent	**5.9**	**519.2**	**2.95**	**259.6**
Primary P removal - particulate P (through solids sedimentation)	-0.4	-35.2	-0.80	-70.4
soluble P precipitation (through coagulation)	0	0.0	3.02	265.8
precipitated P removed in primary	0	0.0	-2.55	-224.4
Composition of primary effluent				
Non precipitated particulate P from raw influent	0.86	75.7	0.46	40.5
precipitated P in primary effluent	0	0.0	0.47	41.4
Soluble P	5.04	443.5	2.02	177.8
P secondary effluent	**1**	**88.0**	**1**	**88.0**
Particulate P	0.18	15.8	0.25	22.0
soluble P	0.82	72.2	0.75	66.0
Non-coagulated particulate P removed through clarification (option 2 & 3) or filtration (option 4)	-0.3	-26.4	-0.34	-29.9
Coagulated particulate P removed (through filtration)	0	0.0	-0.34	-29.9
P removed by cell synthesis + bioP	-4.6	-404.8	-1.27	-111.8
Total Secondary P removal	-4.9	-431.2	-1.95	-171.6
Total P removed	-5.3	-466.4	-5.3	-466.4
Coagulated P removed	0	0.0	-2.89	-254.3
Inlet particulate P removed	-0.7	-61.6	-1.14	-100.3
Soluble P removed without coagulation	-4.6	-404.8	-1.27	-111.8
Total non-coagulated P removed	-5.30	-466.4	-2.41	-212.1
Total P available for recovery downstream anaerobic digestion	-5.30	-466.4	-2.41	-212.1
P recovery potential (downstream anaerobic digestion) as compared to total P removed from the wastewater	**100%**		**45%**	

The table shows that out of the 554 kg/d of P fed to the SEWPCC, 88 kg/d leave with the effluent, so that 466 kg/d are removed by the WWTP. For options 2 and 3, all of the 466 kg/d are available for struvite precipitation, while option 4 allows for recovery of 212 kg/d, which represent 45% of the capacity of option 2 or 3.

NOTE: the Phosphorus recovery business case has not been developed for the 3 options.