Residuals Management Research Team

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Abstract

The Residuals Management sub-team is solving the problem of solids disposal in AguaClara treatment plants. Currently, settled solids from the sedimentation and entrance tanks are drained and routed directly onto the nearby landscape. The newly created stacked rapid sand filtration system will produce backwash water in need of disposal, and spikes of highly turbid influent water bypass the plant by being discharged down the surrounding slope. The research goal is to determine inexpensive and responsible disposal methods for these outflows as well as for precipitate matter removed from the chemical stock tanks. Flow rates and concentrations of all residual flows have been estimated with the help of AguaClara engineers in Honduras, and designs have been created for pipe outlet protection structures which should reduce the erosive power of AguaClara residual flows. The team goal is to identify promising methods and eventually code them into the AguaClara design tool for use in the future and also for possible use in retrofitting current plants.

Background

Current AguaClara plants in operation in Honduras dispose of their aluminum hydroxide/clay residuals by piping them directly to the surrounding countryside. Although no toxicity is anticipated to be associated with this waste material, environmental responsibility dictates that increased attention be given to finding a safe and efficiently engineered solution to the disposal of these materials.

Antonio Elvir, a member of Agua Para el Pueblo and a Honduran AguaClara adviser, has noted that visiting water treatment experts often ask about AguaClara's solids disposal procedure during plant visits. This indicates that responsible disposal is a focal point among those who are assessing AguaClara's success.

Much of the landscape in Honduras is steeply sloped, which makes the problem of preventing soil erosion via residuals disposal difficult. AguaClara is also challenged with the task of preventing the formation of stagnant pools of water, which provide a place for disease-spreading mosquitos to easily breed.

In addition to the alum sludge solids from the sedimentation tank and filter backwash, operational plants are dealing with the following waste streams:

1. Chemical waste from preparation of the chlorine disinfection solution, largely composed of calcium carbonate precipitate
2. High-turbidity influent water which is diverted away from the plant during and after heavy rainfall
3. High-turbidity sedimentation tank effluent water during treatment failure

The ongoing research has these waste streams in mind.

Literature Review

The residuals management sub-team is new to AguaClara, but it is likely that some reworking of an existing sludge disposal method will be the best option for the bulk of these waste streams. Current common unit processes in sludge disposal (in water and waste water treatment) that are potentially applicable to water treatment plant residuals include:

1. Thickening (by gravity or flotation), yielding more concentrated but still pumpable liquid of between five and seven percent solids
2. Conditioning (usually with chemicals), improving dewaterability
3. Dewatering (often utilizing a freeze/thaw cycle)
4. Simple disposal to the surrounding environment

The most important constraints on residuals disposal are considered to be:

1. Legal restrictions on sludge and chemical disposal
2. Affordability
3. Feasibility for future utilization and retrofit
4. Environmental and health impacts

Basic descriptions of these constraints and how they apply to the residuals sub-team follow below.

Legal Restrictions and Standards for Residuals Disposal

Disposal standards differ by information source, but are almost universally vague.

The Ten States Standards (TSS), a well-known starting point for water treatment guidelines, specify that when lagooning alum sludge, the lagoon must be "free from flooding", have a "usable depth of five feet", maintain an "adequate freeboard of at least two feet", and consist of at least two cells such that alternation is possible during failure or routine maintenance[1].

The TSS specifications for mechanical dewatering are very brief and state that use of this practice "depends on the characteristics" of the sludge resulting from "site specific studies"[1].

The TSS rule for land application of alum sludge is especially vague: "Alum sludge may be disposed of by land application either alone, or in combination with other wastes where an agronomic value has been determined and disposal has been approved by the reviewing authority"[1].

The United States Environmental Protection Agency (US EPA) has published a document referred to as "Part 503", that covers the proper disposal of sewage sludge biosolids, and describes multiple alternative methods; unfortunately, Part 503 does not mention similar treatments for drinking water residuals[3]. No EPA documents have been identified detailing the guidelines for correct disposal of drinking water residuals such as alum sludge.

The World Health Organization's (WHO) "Guidelines for Drinking-water Quality", also does not contain any information about residuals management, for either waste water or drinking water[4]. The WHO document instead focuses primarily on the microbial, chemical, and radiological aspects of drinking water treatment.

The Cornell University water treatment plant, operated by Chris Bordlemay, sends its lagoon-dewatered PACl sludge (containing aluminum hydroxide and polymeric aluminum) to the University's grounds department, which dries the sludge even further and then mixes it with regular soil for use in campus landscaping. This practice is allowed under 6-NYCRR Part-160-3.7(b)(4)[14], which is a subsection in the State of New York's rules and regulations specifying that a school that treats its own water is allowed to use its own residuals on site (provided that they are not composted). Cornell also mixed this sludge with soil in a ratio such that levels of aluminum fall below that required or remediated industrial sites, though this is an informal practice and is not officially required.

It is likely that AguaClara water treatment plants could continue to dispose of their settled sludge without engineered management almost indefinitely and likely face no legal obstruction. Most treatment plants in Honduras, even those benefitting from the use of electricity, choose this option. However, this is due simply to the fact that Honduras does not have an established water quality authority that monitors residuals disposal. In anticipation of future national regulations, issues with scale-up, and because expansion of AguaClara will involve other nations, it is prudent for the program to design disposal methods.

Affordability

Cost constraints are embodied in all AguaClara technologies. Residuals management needs to be approached from the same perspective.

Sludge thickeners and conditioners require large storage containers and careful manipulation in order to achieve the desired solids consistencies, and are difficult to operate without electricity. Heat drying of solids is also nearly impossible without a constant, high amount of energy.

Dewatering is a unit process that was initially thought to be a possible choice for managing alum solids. Equipment requirements for the vacuum filter, centrifuge, and filter press methods are very restrictive, so those methods have been eliminated from consideration[6]. Lagooning and bed drying seemed to be viable options until further design issues were identified; this will be detailed in the next section.

Lagoons

Lagooning is the process of piping sludge into an open basin with the end goal of dehydrating the solids via evaporation and percolation. As such, construction costs are not very high. Lagoons require only initial excavation and sporadic solids removal, but otherwise utilize no infrastructure that must be purchased.

However, alum sludge dewaters poorly and often takes months to dry from a ~1% solids content to ~3%[5]. Mechanical equipment is often used to remove the settled solids, adding to the equipment cost. In addition, the rainy season that occurs from April to November in Honduras could add significant volume to the ponds.

The slow dewatering process presents an additional problem in that it provides a place for mosquitos to breed in a body of stagnant water that is likely close to Honduran homes. AguaClara would like to avoid causing any increase in mosquito-borne illness, so this issue is very important when considering lagoons as a dewatering option.

Drying Beds

Drying beds vary in design, but the most common setups involve a lined basin equipped with perforated underdrains and filled with layers of sand and gravel[6]. These materials represent a capital cost and transport requirement, and underdrain decanter installation is another cost. As in the case of lagooning, it is necessary to remove these settled solids; this further adds to total cost.

Therefore, though lagoons appear to be the more feasible dewatering option in terms of construction cost, neither is a particularly attractive option.

Alternative Methods

Another possible disposal method considered was irrigation using the wet alum sludge. This alternative would eliminate the need for storage and dewatering. Irrigation will be detailed in the next section.

If no treatment method is found to be economically viable, it may be necessary to devise a plan for responsible, direct disposal. In this case, AguaClara would continue to discharge its waste streams into the surrounding environment without treatment, but would limit its erosive effects on the land by dissipating its energy. This could be accomplished using winding channels, riprap flow obstacles, or any other structure designed to minimize flow velocity and encourage percolation.

Feasibility in Honduras

AguaClara plants are constructed in areas of mountainous topography and variable soil composition (the Ojojona plant, for instance, is located on top of exposed rock). AguaClara leader Monroe Weber-Shirk has stated that considerably more than half of the Honduran landscape has a slope of over 30% grade. Site characteristics are anticipated to be an issue in terms of what can be constructed for residuals management.

Lagooning requires excavation, which would be impractical if the plant were located above an impermeable rock layer. The area requirement for a lagoon might also be restrictive due to the fact that several AguaClara plants are surrounded by forest and do not have very much extra room for construction. The hilly topography of Honduras is likely to be another constraint on lagoon construction.

Removing the settled solids from lagoons could be difficult because mechanical equipment is not readily available in the remote regions of Honduras and is dangerous to maneuver on hilly terrain. Therefore, settled solids would likely need to be removed by manual labor. After removal, it is also necessary to transport and to find an end location for the dried solids; this would require AguaClara to integrate a method such as land-spreading to dispose of the settled "dry" sludge.

Irrigation

Irrigation was an intriguing unit process because of its low cost and applicability to many different site types. Past studies have established that alum sludge application to land has no apparent negative effects on at least two types of vegetation systems[7][8].

One possible problem associated with irrigation using alum sludge identified in one study and corroborated by Professor Larry Geohring (BEE) is the tendency for aluminum hydroxide precipitates to reduce the amount of soluble phosphorus in the soil[7]. This could potentially cause nutrient limitations that may impact plant growth. Information about Honduran soil qualities such as pH and organic content is being sought to assess this possibility. Honduras does not benefit from the thick volcanic ash that characterizes many rich Central American soils.

The design of an irrigation system could take many forms, and would likely need to change based on each location's topography. A theoretical configuration would include furrows oriented orthogonally to the slope of the nearby hillside, allowing a zig-zag flow path and thus dissipating the erosive energy of the sludge flow. Settled solids from the sludge would accumulate in the furrows, either countering the effects of erosion or causing sediment buildup, which could be remediated by re-plowing.

Research into the native flora of Honduras was difficult. A list of possible plants that could be used in an irrigation system was impractical to assemble due to a lack of available information, and Honduran APP member Antonio Elvir was quick to mention during a meeting that most of the growth occurring around AguaClara plants is weedy and difficult to classify. The alum sludge from AguaClara plants has not been shown to contain toxic pollutants, so edible crops like corn would be a possibility in an irrigated garden setup.

Professor Emeritus Richard Dick, Civil & Environmental Engineering, has studied sludge disposal for years. After being consulted about this possible irrigation method, he was very skeptical of its potential. Alum sludge dewaters very poorly, often taking months or years to lose even a portion of its water. With a constant influent sludge flow and occasional rainfall events, the sludge will likely be continually rewetted and percolate very slowly, possibly never fully absorbing or settling at all.

Honduras does not benefit from a freeze/thaw cycle, which is a natural dewatering process that assists temperate-zone water treatment plants with alum sludge treatment. As such, evaporation and percolation may not be sufficient to ever fully dewater alum sludge in the wet, warm environment of Honduras.

Stochastic analysis of rainfall events will be useful to whichever design is selected. The volume of rainwater being added to any outdoor sludge management system will need to be considered, as will the potential additions of high-turbidity water bypassing the plant or settled water during plant failure. It has been difficult to obtain any reliable meteorological data, however, due to inconsistent and at-times nonexistent record-keeping in Honduras.

Energy Dissipation

Energy dissipation structures are used to reduce the erosive power of water flows, often in cases of damaging runoff. These structures could potentially be used by AguaClara at the main residuals outfall in order to:

1. Prevent soil mobilization

2. Increase the time before high-turbidity residuals reach nearby streams and distribute flows over time, reducing peak flows

3. Allow percolation and evaporative dewatering by spreading out the flow

Common designs placed at pipe outlets include energy dissipators, pipe-outlet protectors, and plunge pools. Professor Todd Walter of Cornell's Department of Biological and Environmental Engineering referred the Residuals Management subteam to potential flow transition structures such as those featured in a document issued by the Maine Department of Environmental Protection[9].

Energy dissipators and pipe outlet protectors vary slightly in design, but both consist of a bed of rock placed directly in the path of flow of the outlet pipe. In an energy dissipator, the pipe outlet is just beneath the surface of the rock bed, while a pipe outlet protector places the pipe completely above the rock bed (figure 1). A pipe outlet protection structure is the more desirable configuration for AguaClara due to the reduced risk of solids accumulation within the residuals disposal pipe.

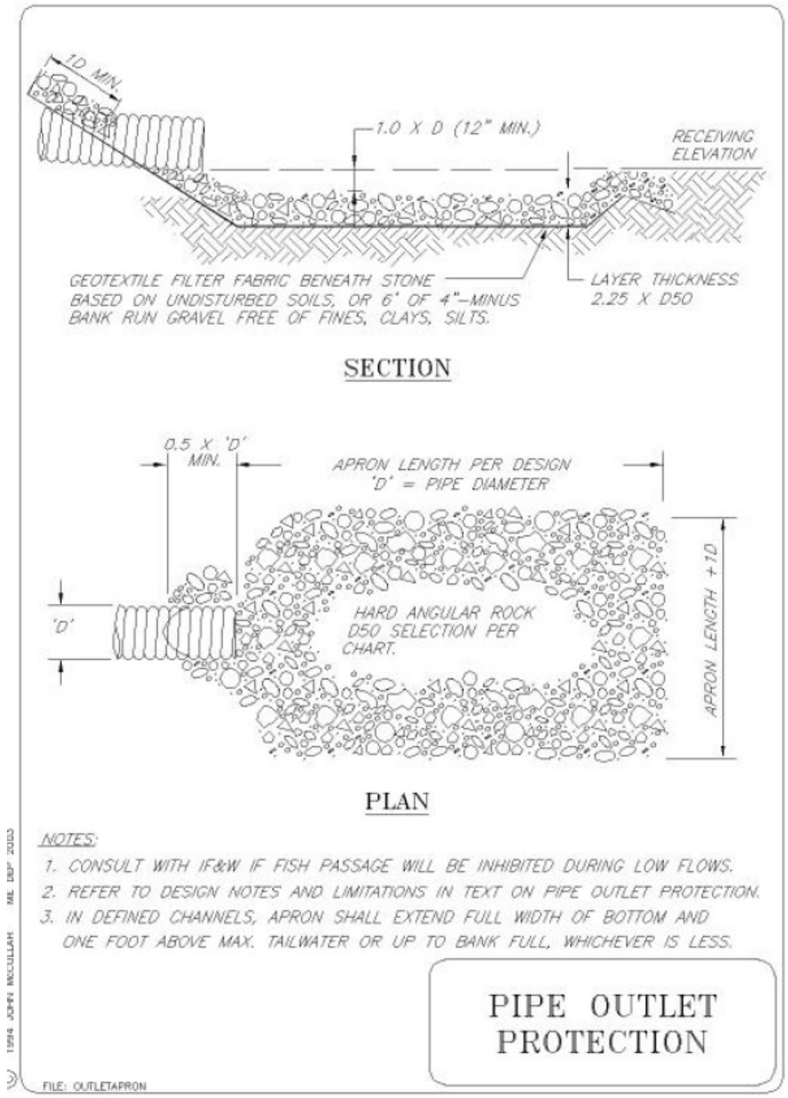


Figure 1: Diagram of a pipe outlet protection structure

After consulting with AguaClara engineers in Honduras, the Residuals Management subteam chose to approximate the mixture of residuals as water, which has been the observed case. Basic design parameters for both structures, such as stone size and bed length, depend on the flow rate of water being deposited into or onto them, as displayed in figure 2.The flow rate of residuals at Alauca, for example, rarely exceeds 70 L/min during tank drainage. This flow rate is far below the lowest specified discharge rate provided by the Maine DOT for sizing of stone, which is 3 cfs (about 5,000 L/min).

OUTLET PROTECTION
FOR A PIPE FLOWING FULL WITH LOW TAILWATER

RIPRAP SIZE - D50 (inches)
PIPE DIAMETER

DISCHARGE	12"	15"	18"	21"	24"	27"	30"	36"	42"	48"	54"	60"
3cfs	4											
5cfs	4											
8cfs	5	4										
10cfs	6	5	4									
12cfs	8	6	6									
15cfs	8	6	8	5								
17cfs		8	8	5								
20cfs		10	10	6	5							
25cfs		12	12	6	6							
30cfs				8	8	6						
40cfs				12	10	8	6					
50cfs				16	12	10	8	6				
60cfs				18	16	12	10	8				
70cfs					18	15	12	8				
80cfs					20	16	15	10	8			
90cfs						18	16	12	10			
100cfs						20	18	12	10			
125cfs						24	20	16	12	10		
150cfs							24	20	16	12	10	
200cfs								24	20	18	15	12

Figure 2: Maine DOT stone size specifications for pipe outlet protection structures.

The highest flow rate that could be experienced at the residuals drain output would be just after the initiation of stacked rapid-sand filter backwash, when residuals flows may approach rates up the three times that of the plant's design flow.

Comparable flow rates occur during simultaneous plant bypass and sedimentation tank drainage. During periods of high turbidity due to rainfall, the flow being diverted toward the plant does not change. This flow is taken from a dam inlet covered by a trash rack and controlled by a valve; thus, it cannot exceed the design flow at each plant. Therefore the flow rate of water being transported directly to disposal during periods when treatment is unfeasible would be on the order of twice the maximum design flow, due to full bypass and full drainage.

Unforeseen flow additions and heavy rainfall, which correlates with high-turbidity plant bypass events, may change the maximum experienced residuals flow even further.

It is likely that the easily-obtainable (and oftentimes left over) stone located at AguaClara plants in Honduras would more than suffice for energy dissipation. AguaClara Engineer Sarah Long indicates the average stone diameter is approximately 12 inches (30.48 cm), with both larger and smaller sizes of stone readily available after routine plant foundation excavation. Additionally, most of the stone is granite and will therefore not be subject to disintegration due to scour.

At least one currently operating plant has already seen the formation of a plunge pool below the sludge drain outlet due to erosion. This example shows that protective action is an important issue and also presents an opportunity for construction of a protective rock bed without needing to excavate quite as much soil as would have been required.

Environmental and Health Impacts

The effect of our chosen disposal method on human health and the surrounding environment is a focal point of the Residuals Management subteam's research.

Human health is the first priority, with potential exposure to plant operators as well as the public being a concern. Toxicity to humans is likely not a concern for the alum sludge in terms of heavy metals or other persistent pollutants, because there are currently no factories or other heavy industry located upstream of AguaClara plants.

It may be necessary to consider the possibility of pathogen contamination in AguaClara's alum sludge. If disease-causing microorganisms are present in the sludge, we will need to take extra care if any of the residuals are handled. However, direct disposal should not present a particular challenge in this regard due to the fact that these microorganisms were originally present in the water body to which they are being deposited.

The calcium carbonate precipitate that results from the mixing of the chlorine solution has a high pH and contains residual hypochlorite ion, which is a strong oxidant. Thus, this small portion of AguaClara residuals is not completely innocuous and should not be directly handled by plant workers.

As mentioned in an earlier section, any residuals disposal practice that creates undrained bodies of water runs the risk of allowing mosquitos to breed and thus increase in population in Honduran villages. AguaClara must prevent this situation from occurring, because diseases such as malaria and Dengue fever are already common in and endemic to Honduras, and outbreaks can cause alarming numbers of illnesses and fatalities. A photographic example (figure 3) of an undesirable situation at AguaClara's Tamara plant below shows the ideal breeding ground that can be created by allowing residuals and plant bypass water to pool near a plant.

Figure 3: A stagnant pool of residual waste water at the Tamara plant in Honduras.

The effect of AguaClara's practices on the natural environment is also of importance. Treatment plants will very likely be disposing of their wastes onto the land surrounding each plant, and as such should be aware of how they might interfere with natural processes. Changes in soil composition due to alteration of pH and organics content may affect the growth abilities of plants, and any sludge that reaches nearby waterways could adversely impact them by greatly increasing local turbidity.

Chlorine residuals could potentially exist in the sludge, and APP's Antonio Elvir has indicated that environmental impacts in nearby riparian habitats are of concern to him. Small flora and fauna could be damaged by the concentrated input of chlorine if care is not taken to dilute or slow this waste stream. However, all current AguaClara plants are located sufficiently far from the receiving body of water such that chlorine residuals will likely take part in oxidation reactions after output, rapidly decreasing in concentration as they interact with organic matter in the soil.

Analysis

Data was collected during the fall semester of 2011, and mathematical analyses were performed. The following information has been gathered about current AguaClara plants, mostly from the experience of Sarah Long, one of the AguaClara engineers in Honduras:

1. Typical design/maximum flows for each plant, currently ranging from 6.3 L/s to 32 L/s
2. Turbidity threshold for plant bypass during high-flow events of roughly 500 NTU
3. Turbidity threshold for sedimentation tank wasting during treatment failure of roughly 20 NTU
4. Granular calcium hypochlorite stock concentration = 53 g/L
5. Typical observed $CaCO_3$ buildup in chlorine stock tank = ~5 cm, or about 33 L per tank
6. Frequency and duration of sludge drainage (more information below)
7. Frequency and current method of solids removal from chlorine solution tanks

Estimation of Alum Sludge Flows

The first step in deciding how to dispose of the alum sludge produced by AguaClara plants was to approximate those sludge flows using turbidity and plant flow data. This can be done in steps using a series of equations.

The approach used, by recommendation from AguaClara engineer Sarah Long, was to go through the calculations using data from the currently operating plant in Alauca. The equations are programmed in a way that will allow the associated files to be used interchangeably to design future plants and possibly retrofit those that are already in operation.

The sludge flow rates were liberally estimated in order to be adequately prepared for high-turbidity events. Therefore upper-threshold values were used for the turbidities experienced by the sedimentation tank and stacked rapid-sand filter.

Estimating Alum Dose

Estimation of the alum dose that should be applied to the influent water is done using the following empirical equation, derived from research conducted by AguaClara contributors[10]:

$$C_{Alum} = 15 + 15 * log(Turbidity) \quad (1)$$

The desired alum concentration in this case is given in mg/L, from a turbidity measured in NTU. The constants in this equation are applicable for temperatures of above 10 degrees Celsius, which should almost always be the case in Honduras.

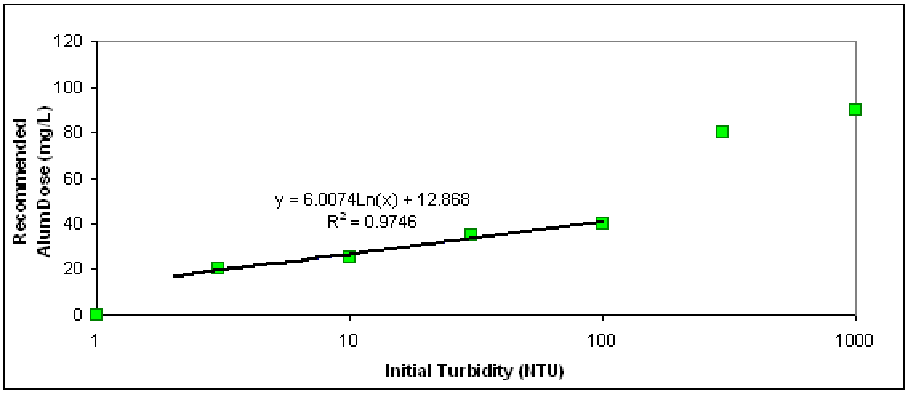


Figure 4: Relationship between influent turbidity and optimal alum dosage

A comparison of the fitted equation to observations (figure 4 above)[10] shows that it does not accurately predict optimum alum dosage above 100 NTU; in fact, the recommended alum dose for 500 NTU influent turbidity appears to be about 80 mg/L. Additionally, Sarah Long and Design team leader Andrew Hart have indicated that 60 mg/L is typically the highest alum dosage applied in AguaClara plants in Honduras. Therefore I assumed that this would be the applied dose in Alauca while experiencing a 500 NTU influent turbidity.

Estimating Aluminum Sludge Flow Rate in Sedimentation Tank

Alum is a hydrated compound with an average chemical formula of $Al_2(SO_4)_3 * 14H_2O$. The fact that each mole of alum contains two moles of aluminum before dissociation should be noted, because each mole of aluminum can be assumed to precipitate as aluminum hydroxide $(Al(OH)_3)$ in a "worst-case" scenario of solids production.

Alum sludge is composed of aluminum hydroxide as well as suspended solids. The amount of aluminum hydroxide formed depends upon the pH of the solution, which influences the amount of aluminum ions that dissociate. These ions and the precipitated $(Al(OH)_3)$ act as a coagulant, sticking to colloidal particles and causing them to settle out of the water.

At typical influent pH values between 5 and 8, the solubility of aluminum species in water is sufficiently low that added alum will form aluminum hydroxide precipitate (figure 5)[12].

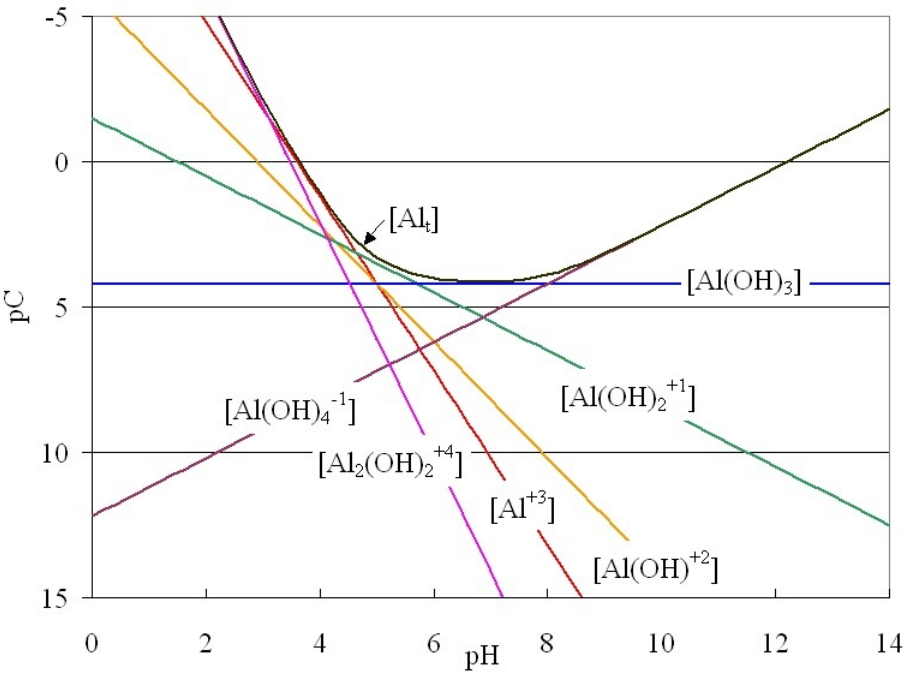


Figure 5: Solubility of amorphous aluminum species in water as a function of pH

Aluminum does not necessarily precipitate with this exact formula ($Al(OH)_3$), and is better described as a polymeric hydroxo-aluminum compound, but it is sufficient to use this formula to approximate the character of aluminum hydroxide precipitate. An upper limit on the total mass of sludge leaving the AguaClara plant was approximated by assuming a maximum influent turbidity value (500 NTU) as well as the maximum settled water turbidity that will result in acceptable filtration after leaving the tank (20 NTU). This is a "worst-case scenario" in which the plant is still operating and treating a very turbid influent to the point at which it is barely able to be filtered.

As a "rule of thumb" 1 NTU of clay turbidity generally converts to a concentration of 1 mg/L. Therefore, the mass component of the settled turbidity contained in the sedimentation tank sludge is known:

$$M'_{Turbidity} = Q_{Plant} * (C_{Influent} - C_{Settled}) \tag{2}$$

Alauca can treat a design flow capacity of 12 L/s, at a maximum influent turbidity of 500 NTU and a maximum settled water turbidity of 20 NTU after sedimentation. These data, when used in equation 2, yield a settled solids mass per time of about 498 kg/day. Note that this does not include the mass of aluminum hydroxide.

The mass of aluminum hydroxide precipitate can be calculated by assuming that all alum dissociates and each aluminum ion becomes part of a settled aluminum hydroxide molecule, as described above. This was done using simple stoichiometry, and at Alauca this yields a total mass per time of about 4 kg/day. Therefore the total mass per day of settled solids from the Alauca sedimentation tanks during a very high turbidity event is about 502 kg. Note that this does not yet include the solids leaving the stacked rapid-sand filter during its regular backwash cycles.

Sarah Long spoke with plant operators in Alauca and determined that they drain all four sedimentation tanks at least twice a day, and likely three or four times during high influent turbidity. She cited a common drainage time (Ti_{Drain}) of 1.5 minutes, which is sufficient to flush out all of the settled solids as well as allow some settled water to dilute the drained sludge before closing the drain valves. This information, along with design information such as sedimentation tank volume, drainage time, and number of drains, can yield total sludge volumes.

First, the average total flow from all sludge drains can be found as a function of each tank's volume and the time taken to completely empty all tanks from their full operational state:

$$Q_{SedSludgeDrain} = \frac{2 * V_{Tank}}{Ti_{SludgeDrain}} \tag{3}$$

The factor of two in equation 3 is due to the fact that average drainage rate is half of the initial drain rate. The flow from each orifice, or individual drain, is then found by dividing the total flow by the number of orifices:

$$Q_{SedSludgeOrificesEst} = \frac{Q_{SedSludgeDrain}}{N_{SedSludgeOrificesEst}} \tag{4}$$

This information, along with the drainage time chosen by the plant operator, yields a volume of sludge per tank (as well as per "drainage event"):

$$V_{SludgePerTank} = Q_{SedSludgeOrificesEst} * Ti_{Drain} \tag{5}$$

Finally,the total volume of sedimentation tank sludge produced during a single drainage event is found by multiplying the sludge volume per tank times the number of sedimentation tanks in use at the plant:

$$V_{SludgeTotal} = V_{SludgePerTank} * N_{SedTanks} \tag{6}$$

Application to Alauca

At Alauca, these calculations produce a volume of about 104 L per drainage, with an average flow rate of about 70 L/min (0.041 cfs) during drainage. Assuming that the tank would be drained four times per day during periods of very high influent turbidity, the total volume of drainage from the sedimentation tank per day is about 417 L.

This volume would yield a mass/mass solids content of about 55% when it is considered that 502 kg of solids are predicted to settle out per day in this scenario.

This value is unreasonably high and predicts that the sludge suspension exiting the sedimentation tank would no longer behave as a liquid. It is known that this "no-flow" scenario has never occurred; it is likely that drainage would occur much more often during a day with influent turbidities this high, if the plant continued to operate. At the time that this report was completed, attempts to contact an AguaClara engineer in Honduras had not succeeded for several weeks. As such, this discrepancy has not yet been explained.

It is likely that the operators are draining the sedimentation tanks for longer periods of time during high influent turbidity episodes. Additionally, most highly turbid influent spikes peak for only periods of time on the order of minutes or hours (due to rainstorms), so this day-long approximation is a hypothetical situation.

Calculations using influent and effluent turbidities of 40 and 5 NTU, respectively, should produce more typical results. Sarah Long has indicated that typical operation of a plant of Alauca's size must result in the sedimentation tank being drained twice per day, which would result in a daily drainage total of about 208 L. The total mass of settled solids per day in the sedimentation tank would be just under 39 kg, leading to an average solids content of about 16% during tank drainage. Again, for the reasons described above, even this value is unrealistic. Actual solids concentrations are likely more on the order of 1% after dilution.

Estimating SRSF Backwash Sludge Flow Rate

AguaClara also needs to know the flow rate of backwash water being contributed to its waste streams by stacked rapid-sand filters (SRSF), the first of which is currently in operation at the Tamara plant in Honduras.

Data and parameters from the SRSF subteam's lab research[11] and preliminary results from the filter in Tamara were used to analyze this portion of AguaClara's residuals output.

For a liberal estimate it was assumed that the SRSF would be removing all of the solids remaining in the settled water after sedimentation. This is not far from reality due to the fact that Tamara's filter has been regularly producing effluent with a turbidity below 0.5 NTU, which is often far below the turbidity of the settled water. The total mass of solids removed per day by the filter is a function of the plant flow rate (all of which is presumably being filtered) and the turbidity of the settled water moving from the sedimentation tank to the SRSF, again assuming that 1 NTU = 1 mg/L:

$$M'_{FilterSolidsPerDay} = Q_{Plant} * C_{Settled} \tag{7}$$

During the laboratory research mentioned above, AguaClara's SRSF subteam found that the solids loading limit for its 4" diameter PVC filter (area = 0.008107 square meters) was 47,000 NTU*L, or 47 g. This is the amount of solids that the filter can typically remove before needing to be backwashed. Assuming that this loading parameter scales linearly with cross-sectional area, the filter capacity for a larger SRSF is:

$$Cap_{FilterMax} = A_{Filter} * (\frac{47,000NTU * L}{0.008107m^2}) \quad (8)$$

The number of times that the SRSF must be backwashed per day is therefore a function of the mass of solids being loaded and the loading limit. I used the ceiling function in Mathcad to round up and get a liberal estimate of the total backwash volume. The ceiling function takes any non-integer value and rounds it up to the next highest integer, in this case yielding the number of complete backwash cycles that will be sufficient to remove all captured solids:

$$N_{BackwashPerDay} = ceil(\frac{M'_{FilterSolidsPerDay} * 1day}{Cap_{FilterMax}}) \quad (9)$$

Given the number of backwash events, the total backwash volume per day can be easily found. The backwash flow rate is the same as the plant flow rate, with settled water being used as backwash water.

$$V_{BackwashPerDay} = Q_{Plant} * Ti_{Backwash} * N_{BackwashPerDay} \quad (10)$$

Initial reports from Tamara suggested an acceptable backwash time of between 10 and 20 minutes in the field, but further results were variable and changes were still being made to the operation of the filter.

Application to Alauca

These calculations yield a total filtered solids mass of just below 21 kg per day when performed using data from Alauca and the continued assumptions of 20 NTU settled influent water and complete solids removal. When a backwash time of 15 minutes is assumed, the equations yield a total daily volume of backwash water equal to 43,200 L. This volume is considerably larger than the volume of solids predicted, because solids content of this waste stream is effectively zero due to the small mass/mass ratio. This may also contribute to the discrepancy in the calculated solids contents of the sedimentation tank sludges. If indeed backwash volumes are this high, effective dilution of both SRSF and sedimentation tank solids would occur throughout operation as has been observed.

Estimation of Chlorine Tank Precipitate Mass

The precipitation of calcium carbonate ($CaCO_3$) formed in the chlorine stock tank is an important process that has been observed to produce up to five centimeters of settled solids per tank, after dissolution of the calcium hypochlorite granules and equalization with the atmosphere. Operators in Honduras have installed drains on the stock tanks in order to periodically remove this white precipitate, often needing to add water in order to loosen it.

The subteam assumed that all of the calcium hypochlorite $(Ca(ClO)_2)$ will be dissociating, which both delivers the chlorine for disinfection and frees an equivalent number of moles of calcium ions. These calcium ions react with carbon dioxide and form calcium carbonate, which settles to the bottom of the stock tank.

The typical calcium hypochlorite stock concentration of 53 g/L converts to a molar concentration of 0.3707M; after dissociation, this is also the molar concentration of calcium ions as well as the molar concentration of calcium carbonate after precipitation. Simple stoichiometry in conjunction with the total stock tank volume of 55 gallons (about 208 liters) yields a predicted total mass of calcium carbonate equal to 7.725 kg (per each filling of the stock tank, and assuming that the tank is mixed well enough to equilibrate with the atmosphere).

The mass of settled calcium carbonate per day during regular operation is then simple to find as well. Assuming that the maximum dose of chlorine is 2 mg/L, this dose can be multiplied by the plant flow to find the mass of hypochlorite solution required per day. This amount can then be divided by the stock concentration of calcium hypochlorite in the stock tank, giving the required flow rate out of the stock tank.

This flow rate can be used to find the volume of stock concentration used per day; this volume gives the fraction of a stock tank exhausted per day, which can finally be multiplied by the mass of calcium carbonate produced per tank to find the average mass of settled calcium carbonate produced per day.

However, the settled calcium carbonate in the chlorine stock tank is only removed when new mixtures are made, so it may be more useful to multiply this final value by the number of days for which is tank is used before it is emptied. Sarah Long indicated that dosing may increase during periods of high turbidity, during which more calcium carbonate than normal would be produced.

Application to Alauca

Regular plant operation at Alauca according to the procedure described above would produce an average of 1.452 kg of settled calcium carbonate per day. This amount is in addition to the small amount of water that the operators must add in order to loosen the precipitate and allow it to flow out of the lower drain valve on the stock tank.

However, as mentioned above, this solids production is better approximated as a spike input of about 7.7 kg calcium carbonate experienced about once per every five days when the tank empties and a new solution is mixed.

Design of Pipe Outlet Protection Structure

Municipal public works and transportation departments in the United States usually design pipe outlet protection structures using a process that utilizes several standard empirical nomographs. These graphs are used to determine several key parameters such as basin length, width, and depth, but are not useful for automated design in their printed form because of the need to hand-pick values. Because of this, it is necessary to determine the governing equations behind each step of the process.

The design procedure compiled by the Urban Drainage & Flood Control District in Denver, Colorado[13] was used in conjunction with governing pipe flow equations from a fluid mechanics textbook[14]. One assumption that was made in order to use this design process was the approximation of the residuals flow as regular culvert drain water, which should be valid considering that maximum flows will occur during plant bypass and drainage, when solids content is below 1%.

The first step in the design process is to confirm that the "low tailwater" condition applies at the outlet of the drain pipe. For all current AguaClara applications and likely all future locations, the residuals drain pipe will drain onto a spread-out area instead of a small waterway, so the condition should always apply. The actual condition is satisfied when the depth of water does not pool to a depth of more than one-third of the pipe diameter (typically six inches) at the outlet. Because the outlet pipe exit will be elevated above the stone layer and drain water will be emptied at the opposite end of the drain structure, we can assume that this constraint is satisfied.

Designing for a Known Residuals Pipe Slope and Diameter

If the slope of the residuals disposal pipe is known, as in the case of an existing AguaClara plant, the design process can be carried out to a fairly high degree of accuracy via the following equations.

Manning's equation can be used to find the maximum gravity-driven flow rate of water possible through the pipe:

$$Q_{DrainMax} = 2.129 * \frac{1.486}{n} * (R_{HydraulicFull})^{\frac{8}{3}} * (S_0)^{\frac{1}{2}} \tag{11}$$

In this equation, n refers to the Manning's coefficient (which depends on friction), $R_{HydraulicFull}$ is the hydraulic radius (in feet) at full flow, equal to the diameter of a circular drain divided by four, and $S_0$is the slope of the drain pipe. This version of Manning's equation returns maximum pipe flow flow in cubic feet per second.

The depth of the water in the pipe during maximum design flow (for a particular plant) can then be calculated based on the ratio of the design flow to the the maximum pipe flow, or $Q_{Design}/Q_{DrainMax}$. The unknown depth in the pipe during design flow is related to the central angle in the pipe as follows:

$$\theta_{Central} = acos(\frac{R_{Drain} - d_{DrainPartial}}{R_{Drain}}) \tag{12}$$

In this equation, $R_{Drain}$is the radius of the pipe, and $d_{DrainPartial}$is the depth of water in the pipe during design flow. The central angle then determines the flow velocity:

$$V_{DrainPartial} = \frac{1.486}{n_{Manning}} * [\frac{R_{Drain}}{2} * (1 - \frac{sin2\theta_{Central}}{2\theta_{Central}})]^{\frac{2}{3}} * (S_0)^{\frac{1}{2}} \quad (13)$$

This in turn can be used to find the partially-full design pipe flow via $Q_{DrainPartial} = V_{DrainPartial} * A_{DrainPartial}$, where:

$$A_{DrainPartial} = R_{Drain}^2 * (\theta_{Central} - \frac{sin2\theta_{Central}}{2}) \quad (14)$$

At this point, the ratio of design flow to maximum pipe flow can be used to find the actual depth of water in the pipe during partial flow, by matching it to the closest depth in a matrix of $Q/Q_{DrainMax}$ratios for many values of flow depth. A computer program is used to carry out this process and a representative graph is included below:

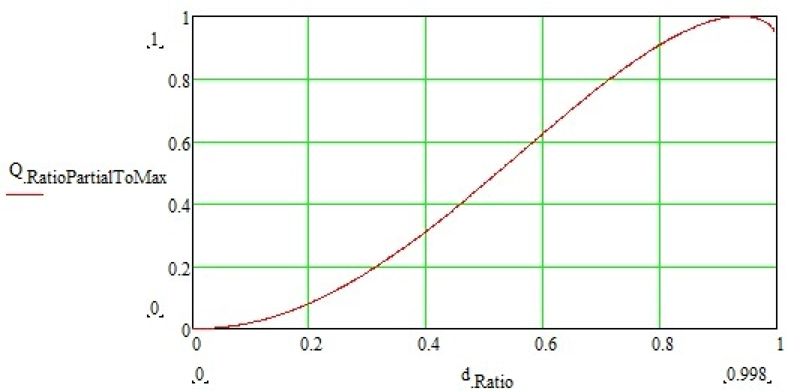


Figure 6: Graphical representation of the relationship between the ratios of design flow to maximum flow and flow depth to pipe diameter in a circular pipe.

The selected depth can then be used to return to the earlier equation 14 and find the cross-sectional area of the flow at the outlet. Velocity at the outlet is then found by simply dividing the design flow rate by the cross-sectional area at the outlet. An additional graph is included below, showing the relationship between the ratio of partially-full flow area to full cross-sectional area and the ratio of flow depth to pipe diameter:

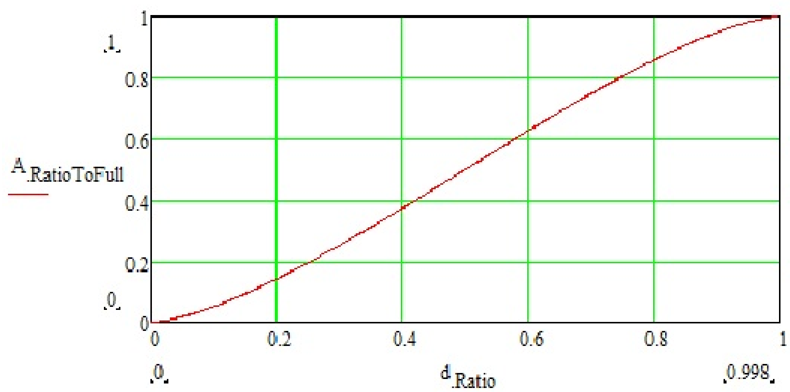
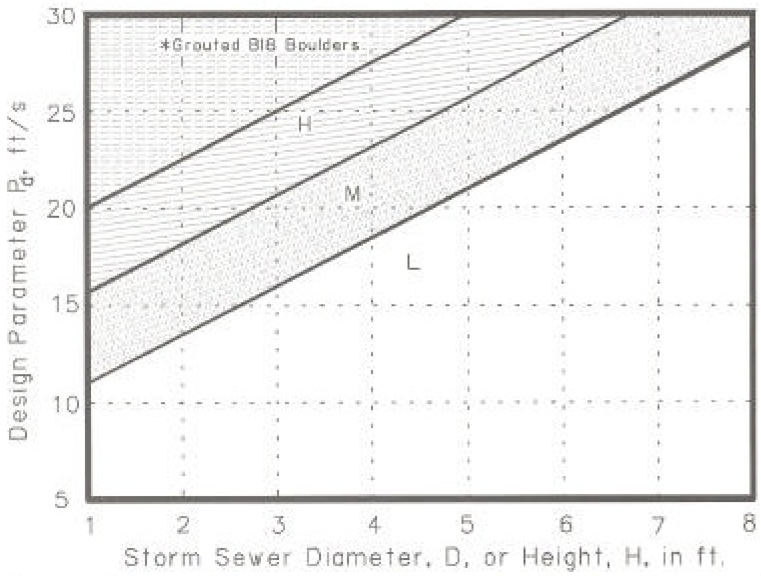


Figure 7: Graphical representation of the relationship between the ratios of design flow area to pipe cross-sectional area and flow depth to pipe diameter in a circular pipe.

The next step in the design process is to calculate the empirical riprap sizing design parameter, which outputs the minimum size of rock needed in the basin to prevent movement or erosion. The parameter is calculated as follows:

$$P_d = (V_{DrainPartial}^2 + g * d_{DrainPartial})^{\frac{1}{2}} \tag{15}$$

, where g is the gravitational acceleration $(32.2\frac{ft}{s^2})$ and the flow velocity and depth are in English units such that the riprap design parameter is in units of $\frac{ft}{s}$. This parameter is used in conjunction with an empirically-derived graph (again, coded into an automated program) in order to determine the proper riprap size:



Riprap Type	D_{50} - Median Rock Size (Inches)
L	9
M	12
H	18

Figure 8: Graph and table used to properly size riprap in a pipe outlet protection structure, as a function of drain pipe diameter and riprap sizing parameter.

Once median riprap size has determined by the above procedure, the minimum required thickness and width of the riprap layer in the protection structure are calculated by:

$$T_{Min} = 1.75 * Size_{Riprap} \tag{16}$$

$$W_{Min} = 4 * D_{Drain} \tag{17}$$

, where $D_{Drain}$is the diameter of the drain pipe. As can be easily seen, the width of the basin only depends on the diameter of the drain pipe. This likely suffices for usage with large culverts, but a factor of safety will likely need to be added due to the small size of AguaClara residuals drain pipes. Finally, the following two equations are used to calculate the minimum required length of the riprap basin, with the larger value generally picked as a safety factor:

$$L_{Min} = W_{Min} \quad (18)$$

$$L_{Min} = (D_{Drain}^{\frac{1}{2}}) * (\frac{V_{DrainPartial}}{2}) \quad (19)$$

The second term is another empirical expression, accepting values in units of ft and $\frac{ft}{s}$ and returning a length in feet.

Designing for an Unknown Residuals Pipe Slope and Diameter

Alternatively, liberal estimates for pipe outlet protection structure dimensions can be carried out without knowing the actual slope of the residuals disposal pipe.

Different AguaClara plants will need to have varying residuals pipe diameters depending on the maximum flow rate expected to pass through them at any given time over the course of plant operation. This maximum flow would likely occur during a period of time when the plant is emptying all of its sedimentation tanks while influent flow is also bypassing the plant due to untreatable turbidities. This flow would be equal to twice the plant's design flow, and it would be safest to size the structure assuming an additional factor of safety, in which the maximum residuals pipe flow is three times the plant design flow.

The first step in this design case would be to determine the pipe diameter required to accommodate this maximum flow. This can be done by assuming that minor losses will be negligible, because the drain pipe will likely have little no bends, expansions, or contractions.

Then, if it is also assumed that head loss in the system is equal to the loss of elevational head, the required diameter can be found one of two ways, depending on whether flow in the pipe is laminar or turbulent. The dimensionless Reynolds number at which transition from laminar to turbulent flow occurs is 2,100.

If flow in the drain pipe is found to be laminar, minimum pipe diameter can be found by the following Hagen equation:

$$D = (\frac{128 * \upsilon QL}{gh_f\pi})^{\frac{1}{4}} \quad (20)$$

In the preceding equation, nu represents the kinematic viscosity of water, Q the maximum design flow, L the total length of the drain pipe, and $h_f$the change in elevation from the inlet of the drain pipe to the outlet.

If flow in the pipe is found to be turbulent, minimum pipe diameter can be found via the following Swamee equation:

$$D = 0.66 * [\varepsilon^{1.25} * (\frac{LQ^2}{gh_f})^{4.75} + \upsilon Q^{9.4} * (\frac{L}{gh_f})^{5.2}]^{0.04} \quad (21)$$

In the preceding equation, all symbols represent the same parameters as in equation 20, with the addition of epsilon, which represents pipe roughness and is equal to around 0.0015 mm for commercially-drawn PVC tubing.

In most cases, the diameter of pipe required will not be the same as the actual inner diameter of commonly-available PVC pipe sizes. In this case, the size of PVC pipe with the next-highest diameter should be selected so as to accommodate the entire flow. This can be done automatically using a pre-loaded array of available Schedule 40 PVC pipe sizes.

The next step in this "worst-case" design process is to find the maximum velocity of water that could possible come out of a drain pipe with this slope and diameter, which occurs when water depth in the pipe is 81.3% of the pipe diameter and is equal to:

$$V_{DrainMax} = 0.718 * \frac{1.486}{n} * (R_{HydraulicFull})^{\frac{2}{3}} * (S_0)^{\frac{1}{2}} \tag{22}$$

Parameters and units in this equation are the same as in equation 11, and the output units are again in $\frac{ft}{s}$. This outlet velocity and the associated depth (as mentioned above) can then be used in equation 15 to size the riprap needed, with the pipe outlet protection structure dimension calculations following as in the "known slope" method.

Application to Alauca

In using the "known slope" design procedure to approximate the specifications of a riprap pipe outlet protection structure for the AguaClara plant in Alauca, I made a few assumptions. I first assumed that the drain pipe would have a slope of around five feet per every one hundred longitudinal feet; this is not completely flat and allows for a margin of error. I also assumed a Manning's coefficient of 0.005, which corresponds to typical circular PVC pipe with a smooth inner surface. An additional assumption I made was to assign a nominal diameter of six inches for the drain pipe, which was cited by AguaClara engineer Jeff Will as a typical drain pipe diameter in current plants.

These inputs returned a pipe flow depth of 2.59 inches, which yielded an outflow velocity of 15.55 feet per second. These values corresponded to a median riprap size of 18 inches laid to a thickness of at least 32 inches. The riprap basin itself was calculated to need minimum dimensions of about two feet (width) by just over five and a half feet (length).

Using the "worst-case slope" method at Alauca resulted in a required nominal PVC pipe diameter of four inches (the current actual nominal drain diameter at Alauca is six inches, due to the actual slope being much less steep than 30%), with a flow depth of 3.27 inches and an outflow velocity of about 35.5 feet per second. This led to a recommended riprap diameter of 30 inches, which is the largest size that can be predicted by this method and may be oversized due to scaling issues with the design procedure used (it is meant for larger flows than AguaClara typically encounters).

The minimum structure dimensions recommended by the "worst-case slope" method were a depth of about 53 inches, a width of about two feet (which should likely be at least doubled as a factor of safety), and a length of about 13 feet. It is likely that a design somewhere in between these sizes and those calculated with the "known-slope" method would be cost and labor-effective, with more than enough energy-dissipating capacity.

Construction and Placement Considerations

At Alauca, and at most other AguaClara plants, these pipe outlet protection structures will likely need to be placed in an excavated portion of the hillside, such that the basin can lay flat. Thus less piping will be needed, as the pipe will terminate at the inlet end of the riprap basin. However, the next design problem lies in the prevention of flow reconcentration at the "far" end of the riprap basin, above the slope. If flow is allowed to exit the basin and reconvene, gravity will result in almost as much flow energy as if there had been no protection structure at all, and erosion will still occur.

One possible exit design would feature several V-notch cuts at equal level such that numerous small flows would exit the basin and be discouraged from joining. Evenly-spaced riprap at the low end of the structure could achieve this desired flow division effect, with small excavated channels delivering the individual rivulets to the receiving body of water.

It is also possible that the bottom of the structure be oriented parallel to the slope, with the entire structure thus being situated on an angle. As long as riprap were placed at the upper end of the structure, in order to reduce erosion, this may be possible.

Residuals flow will be piped from the plant to a distance as close to the receiving water body as possible, so that possible erosive effects after reconcentration do not damage a large portion of the hillside. The pipe outlet protection structure should be placed above the high water level if located near a stream, so that possible flooding events do not lead to washout. This water level may be known by Honduran locals, if there are no associated records.

Effect of Coagulant Change

AguaClara is using polyaluminum chloride (PACl) instead of alum at the 4 Comunidades plant in Honduras. This change in coagulant is being done due to the apparent increase in effluent clarity seen when using PACl.

Preliminary discussions with Po-Hsun Lin, Cornell post-doctorate, have indicated that PACl does indeed capture more particles. He also cited Chris Bordlemay (manager of the Cornell water treatment plant) as noting that PACl produces a higher amount of sludge than does alum. Bordlemay mentioned that this PACl sludge dried out even more slowly than alum sludge, and that the treatment plant actually purchased an expensive organic anionic polymer to assist in the sludge drying process.

According to Lin, the main difference in sludge composition between alum and PACl is that PACl sludge is made up of polymeric aluminum as well as aluminum hydroxide and has a more compact consistency than does pure alum sludge, which is light, fluffy, and amorphous.

Conclusions

Although nothing definite has been decided, some management methods have been eliminated as too costly or inappropriate for the terrain and plot size of a typical AguaClara plant. The devised system will need to accommodate large and unpredictable flows, as well as a range of influent turbidities and characteristics. Sludge irrigation initially appeared promising, although further research seemed to reduce the apparent viability of this option.

Energy dissipation has emerged as the most favorable route for dealing with residuals at AguaClara plants, due to difficulties with actual treatment. Pipe outlet protection structures can be designed using existing procedures, and may be able to reduce flow energy to the point where erosion will not be a factor underneath the residuals drain pipes.

Future Work

CEE and BEE faculty have provided and continue to provide additional insight into this issue. Among them are Professor Leonard Lion (CEE), Professor Larry Geohring (BEE), Professor Todd Walter (BEE), Professor James Gossett (CEE), Michael Rolband, P.E. (CEE Advisory Council), and Professor Emeritus Richard Dick (CEE). The Residuals Management subteam has also received help from Chris Bordlemay, head of the Cornell Water Filtration Plant.

Current governing equations for pipe outlet protection structures have been coded into Mathcad, and need to be edited so that they can potentially be integrated into the design tool.

Once an appropriate design has been selected by the procedure described in this report, it may useful to construct a pipe outlet protection structure at an existing AguaClara plant (preferably with a low design flow). Its performance can then be analyzed through communication with engineers in Honduras, and its potential utility gauged such that AguaClara can decide whether to move forward with additional structures.

If a design is found that will perform as needed across multiple applications, it will be added to the design tool such that users in other countries can be sent specifications for energy dissipation structures along with designs for their AguaClara treatment plants.

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