Water System Design for Wasa Village

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Executive Summary

The goal of this project is to design a water distribution system to provide the village of Wasa with clean, potable water. Wasa is located about 60 kilometers southwest of Iringa and has a population of approximately 3000 people, or 480 households. Wasa is a sprawling village with six sub-villages that contain a primary school, secondary school, teachers college, two dispensaries, a Lutheran church, as well as a large Catholic church and mission. Geographically, Wasa is relatively hilly, with some sub-villages lying in valleys and others on hillsides. The total elevation profile varied by approximately 65 meters, making a gravity-fed water distribution system feasible.

Wasa currently has two gravity fed water distribution systems that extend over four sub-villages and a spring source that has ample capacity. The current system has a cistern that is open to the environment and flush with the ground, resulting in a system that is susceptible to pollution due to runoff. In order to provide Wasa with clean water, a two-phase approach was conceived. It is proposed that Phase 1 of the project would involve building a settling tank adjacent to the cistern that is currently feeding the main line of the village's existing system. The new settling tank would be designed to filter out contaminants, and would be covered and have raised walls to eliminate contamination from surroundings. From this new and improved settling tank, a gravity-fed line will extend 2.4 km into the sub-village of Nyamagola B and provide water to the 51 households that reside in the valley. Also in Phase 1, it is proposed that two hand-pump wells would be mud-rotary drilled in the sub-village of Uhepwa. Uhepwa consists of approximately 52 households and is unreachable from the spring source that feeds the existing main line and proposed new line. Phase 2 of the project is to connect the existing distribution points to the line constructed in Nyamagola B during Phase 1, so that the main line would be supplied with clean water from the updated settling tank.

With the design of this new system, the village leaders of Wasa have agreed to establish a money collection system that will fund the upkeep and repair of their systems. The total estimated cost for Phase 1 of the project is $22,891, $5,316 of which is expected to be in kind contribution. Phase 2 of the project will cost very little compared to phase one, being that Phase 2 involves only the construction of a connection point, using a small distance of pipe. Phase 1 of the design is expected to have a high value of in-kind contribution, as approximately 2.6 kilometers of pipe needs to be dug 1m deep into the ground. A project design best serving the needs of Wasa will be finalized with the help of Saint Paul Partners and their continued communication and collaboration with the village of Wasa.

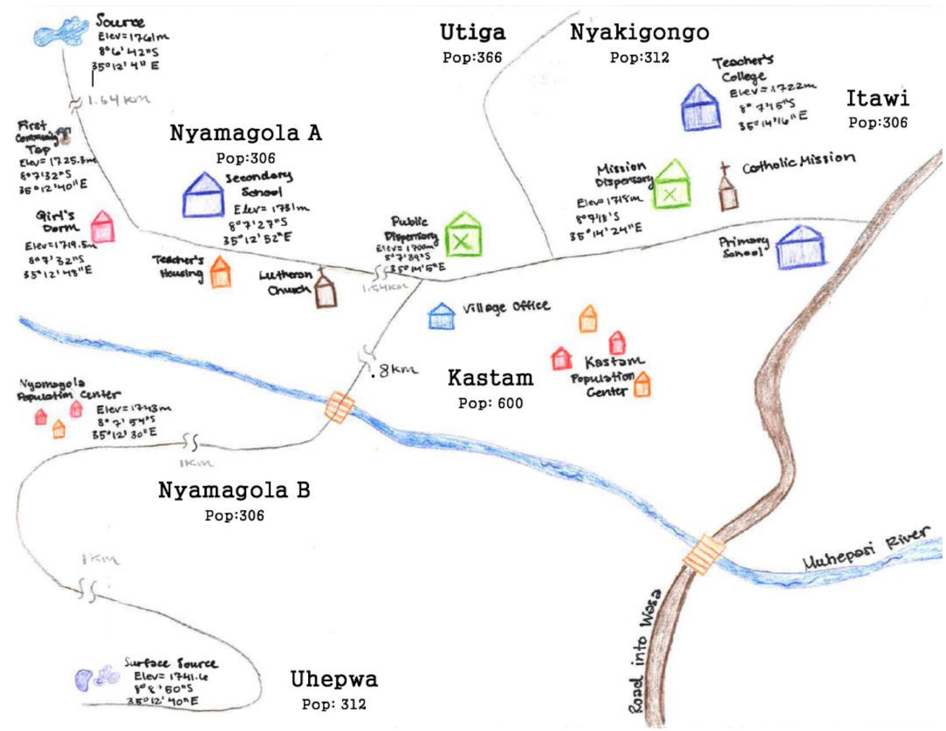

Figure 1: A hand-drawn map of Wasa including the sub-villages and location of current water system and public taps.

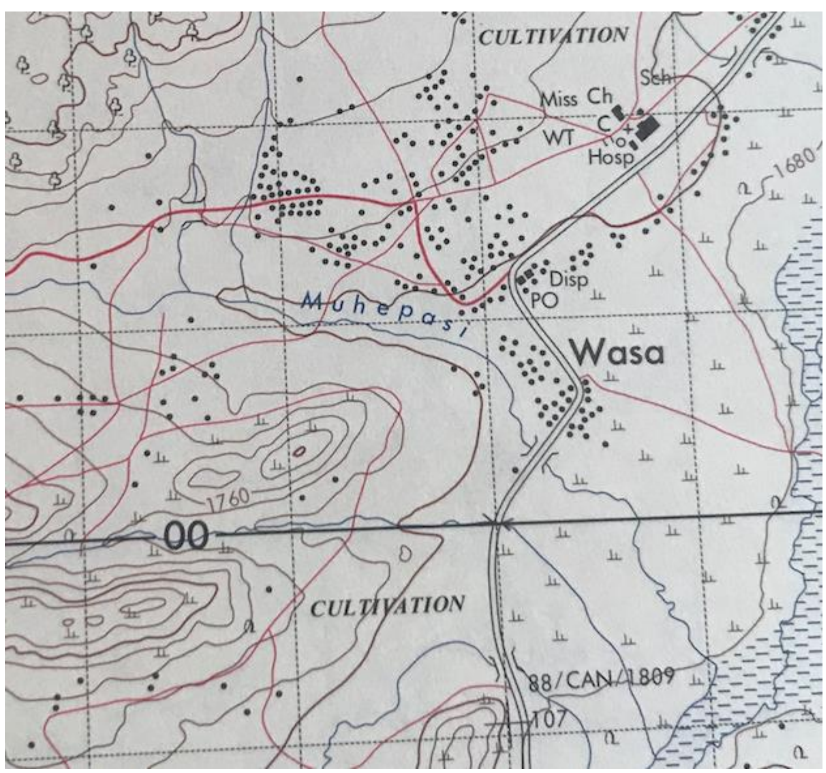

Figure 2: Topographical map of Wasa.

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Background

Wasa village is located approximately 70 km southwest of Iringa Town, were the Lutheran Center, Bega Kwa Bega, and necessary resources are located. The roads heading into Wasa from Iringa are dirt roads, but are very wide and flat, and should not pose an issue for transportation of materials and machinery. The village itself is located in a low lying area surrounded by rolling hills, and the residences of the villagers sprawl for many kilometers. Wasa has a population of 430 households, or approximately 3000 people, which is divided into six sub-villages, locations and populations of which are displayed in *Figure 1* and *Table 1*. Though most of Wasa residents make their living by farming, the village also features two dispensaries, a primary, a secondary school, and the teachers college, where Tanzanians go to learn how to educate.

Table 1: Sub-village populations given by water committee members.

Subvillage	Number of Households	Number of Residents
Kastam	100	600
Nyamagola	102	612
Itawi	51	306
Nyakigongo	52	312
Utiga	61	366
Uhepwa	52	312

During the team's visit, meetings with the water committee revealed village priorities. After much deliberation, the water committee decided they would like to focus efforts on the sub-villages of Uhepwa, and southern half of Nyamagola, which the villagers referred to as Nyamagola B. Nyamagola is divided into Nyamagola A and B by the Muhepasi river. The two focal points can be seen encircled in red in *Figure 3*. Uhepwa is home to approximately 312 people, and Nyamagola B is home to approximately 306 people.

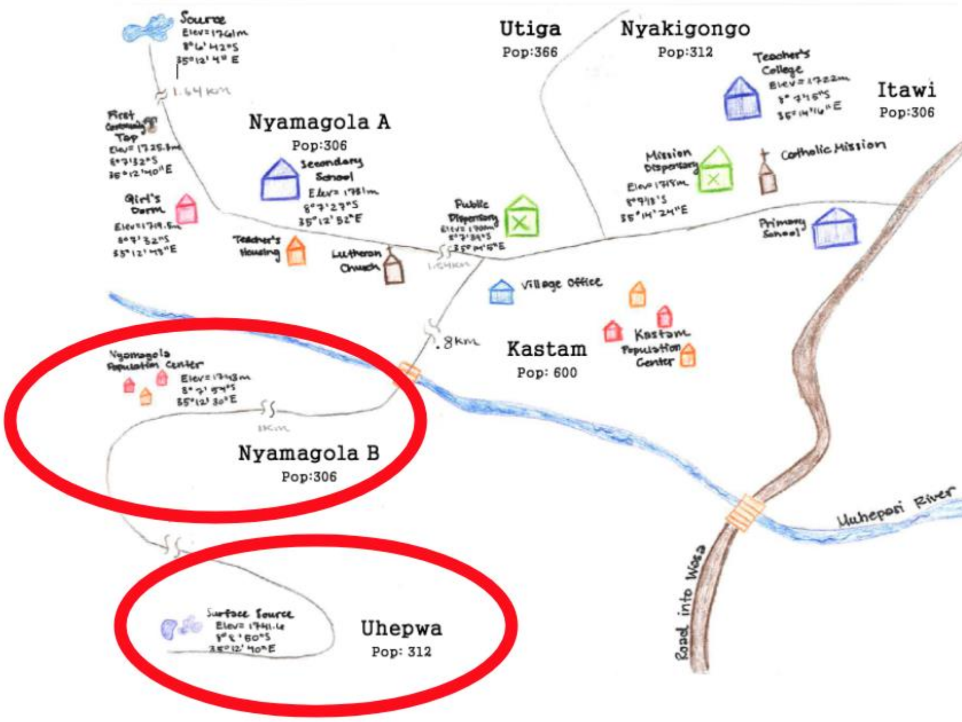

Figure 3: Map of Wasa with highlighted priority areas.

Wasa village has a water system in place, but it does not serve either of these priority areas. The subvillage of Uhepwa is currently gathering water from two small ground sources, seen in *Figure 4* .These sources are believed to be spring fed, and are reported to have a recharge rate of approximately 15 minutes. Residents of Nyamagola B fetch water from the Muhepasi river, displayed in *Figure 5.*

Figure 4: Photograph of 1 of 2 Uhepwa ground sources.

Figure 5: Photograph of Muhepasi river source.

The water committee requested that a line from their existing system be taken across the river to Nyamagola B, and that hand pumps be installed in Uhepwa. The water committee feels that Nyamagola B is reasonable to be reached with their current system, but Uhepwa is too far away and faces too large an elevation challenge to be reached with the existing source. Nyamagola B is approximately four kilometers away from the current source. The elevation profile running from the current source of the main line to Nyamagola B and Uhepwa can be seen in *Figure 6.*

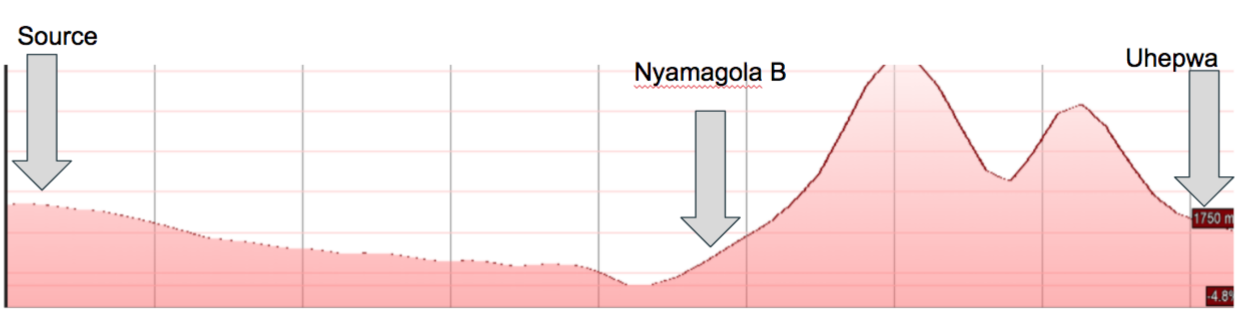

Figure 6: Elevation profile from source to water committee priority areas.

Current System

The village of Wasa currently has two water distribution systems. There are no hand pumps, and there has never been an attempt to drill. Other sources utilized by villagers include the Muhepasi river, small streams, and springs. The first system, which will be referred to as the "Mission Line", was built in 1965 by the Italian Catholic Church and rebuilt in 1991. The Mission Line services the subvillage of Itawi as well as the Catholic Mission and Primary School and is fed by a spring source. The other water distribution system, which will referred to as the "Main Line" is also fed by a spring source that goes into a cistern and was built in 2007 by the Italian Catholic Church. The general geography of both of these systems is shown in *Figure 7.*

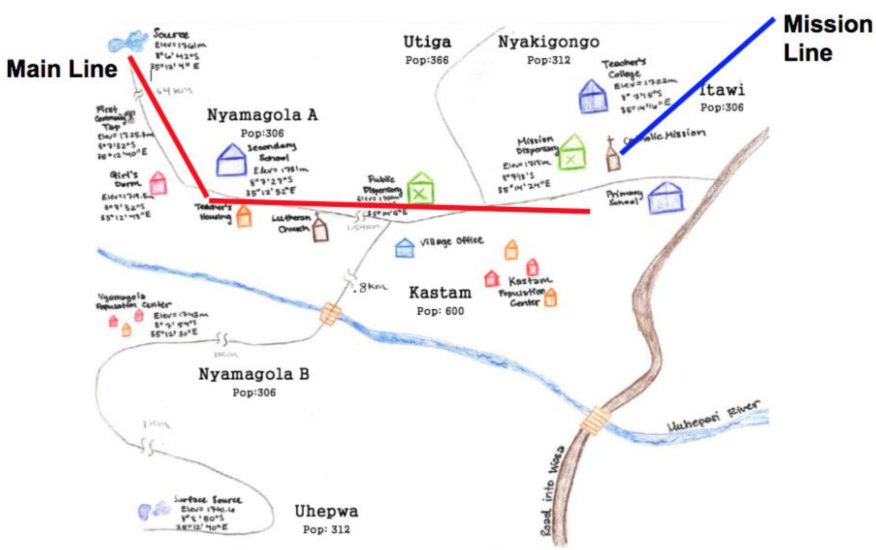

Figure 7: Overlay of existing "Main Line" (red) and "Mission Line" (blue) on hand drawn map of Wasa.

The Main Line is about 5.44 kilometers of pipe and services the sub-villages of Custom, part of Itawi, Utiga, and Nyamagola A. In total there are about 13 public taps and many more private taps (see *Figure 8* for reference).

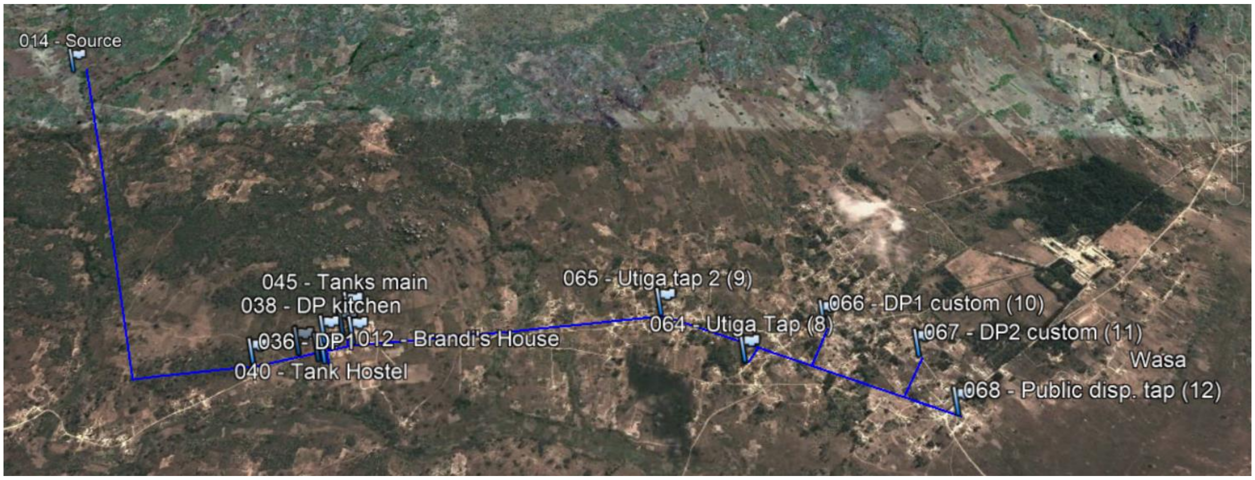

Figure 8: Google earth image of current public distribution points on main line.

The public taps service the secondary school, the teachers in the secondary school, the dormitories where students stay, a dispensary, and the general public. The current state of the Main Line is suboptimal; the line is riddled with burns, cuts, leaks, and bursts due to the improper burial of the pipe and its exposure to the sun, agriculture, and mischievous children. An example of the type of damage this pipe has sustained can be seen in *Figure 9*. The village leaders have been unable to fund the repair of the system since Wasa is not currently charging their villagers for water. Their current system was gifted to them by the Catholic Church, who believed that everyone should be able to collect water for free, and requested that they not charge any fees. However, the village leaders have expressed that they would like to have a payment requirement for their system.

Figure 9: Burnt pipe observed on gravity main running through farmland in Nyamagola A.

Despite all of the damage to the system, the flow rates from the taps are still relatively strong and steady; the flow rate tests taken by the team can be seen in Figure 10. When the team took samples from the source and from the taps on the Main Line, it was found that the source was very contaminated. Results of the water test can be seen in Figure 11. However, samples taken from the taps prove to be less contaminated but still not clean.The test was taken on a day where it was raining heavily and so the rainwater and agricultural runoff into the cistern were believed to affect the bacteria test of the source. As shown in *Figure 13*, the cistern that the source feeds into is flush with the ground and lays in an area surrounded by farmland, causing a lot of runoff contamination into the cistern. A physician at the public dispensary reported that approximately 50% of all patients treated are suffering from typhoid or gastrointestinal related issues. This data further reinforces that their current water sources are unsafe to drink.

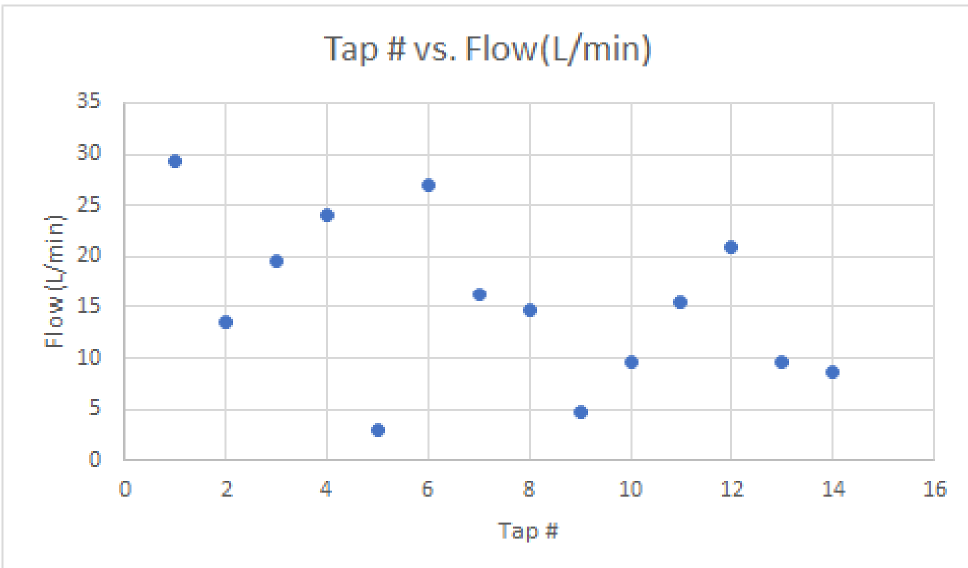

Figure 10: Flow rate measurements of distribution points on the main line.

Figure 11: Contamination results of spring source.

Figure 13: Spring source of main line system.

Village Needs

At the end of the water committee meeting, the design team came away with three water distribution priorities for the village. The first priority was to get water to the sub-village of Uhepwa, where people were getting water out of small springs in the ground or muddy rivers. One of the Uhepwa sources can be seen in Figure 14. The second priority was to expand the Main Line to the other half of Nyamagola, which will be referred to as Nyamagola B. As a final priority, the water committee wanted to further expand off the Main Line and add two more taps in the sub-village of Custom.

To further understand how the village's priorities could be fulfilled, data was gathered about the sub-village of Uhepwa. Uhepwa is approximately one and a half kilometers from the sub-village of Nyamagola B and varies greatly in altitude, which can be seen in Figure 6. The team concluded that the Main Line could not reasonably be extended into the sub-village of Uhepwa. For this reason, the decision was made to include two hand-pump wells located in Uhepwa in the design. Uhepwa is very flat and lies in a relatively wet valley where mud-rotary drilling has the potential to be successful in servicing the majority of the sub-village.

Figure 14: Photograph of 1 of 2 Uhepwa ground sources.

Nyamagola B has approximately 52 households, and is located on the side of a valley. The Muhepasi river, at the bottom of the valley, divides Nyamagola B from its Northern counterpart, Nyamagola A. The elevation change between the main line source, which is located in Nyamagola A and the population center of Nyamagola B can be seen in Figure 6. Nyamagola A is serviced by the main line, whereas Nyamagola B is not, so it was a top priority of the water committee to get water to Nyamagola B. The implementation of a water distribution system in Nyamagola B would allow people to safely obtain water for farming and other domestic tasks instead of walking a long distance down to an unclean river source.

The last priority established by the water committee was to expand the Main Line with two more taps in the sub-village of Custom so that they could service more people. Custom contains about 100 households and there are currently only two public taps in Custom to service the people that live there. The flow rate coming out of the spring source of the current system is large enough could easily be expanded to add two more distribution points, however, since the village leaders seemed to have a good understanding of how to accomplish this independently, this priority will not be included in the design.

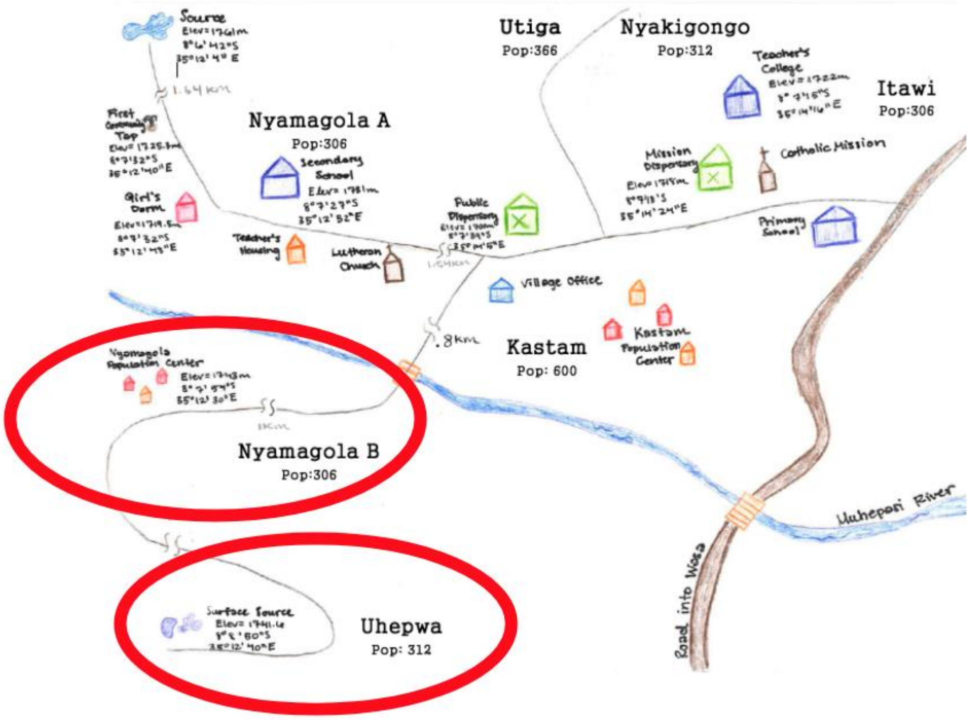

Figure 15: Hand-drawn map of village priority areas.

Tanzanian Design Criteria

Water demand should be based on 25 liters per person per day (25 L/p/d). For schools the design should be for 10 liters per student per day.

Since the system in Nyamagola A will not be redesigned, the flow rates there were considered to be satisfactory, because they already meet the village needs. These flow rates were measured and used in the calculations for designing the new system. However, two distribution points were designed for the system in Nyamagola. The first was located roughly in the center of the Nyamagola B population distribution. Since the village leaders estimated that there are 52 households and 6 people per household is a reasonable assumption, the population in Nyamagola B was approximated at 312 people. The second distribution point was next to the river and less dense in population. Considering the relatively small number of households, perhaps 10 - 15, a conservative estimate of 100 people was taken. Therefore, with roughly 412 people to be accounted for at 25 L/p/d, the Nyamagola system was designed for a total flow rate of 7.2 L/min; 5.4 L/min delivered to the main population center and 1.8 L/min to the distribution point next to the river. The two schools in the village were already serviced by the existing systems, so they were not considered in these calculations.

The design period should be for a minimum of 10 years. Recent population data should be inflated at a rate of 1.5% per year. This means that all designs should design for a 16% population growth, i.e. (1.015)^10.

Once again, the existing system was not considered since it will not be redesigned. However, the Nyamagola flow rates calculated in the above section did not take population growth into account. Thus, the calculated flow rates needed to still be multiplied by 1.16 their previous values. This resulted in flow rates of 8.3 L/min, 6.3 L/min, and 2.0 L/min for the total system, the population center distribution point, and the river distribution point, respectively.

The system should be designed to accommodate 2.5 times the average rate of demand. Hourly water demand is bimodal, with the largest peak in the morning, followed by a lull around noon, and a second peak in the late afternoon.

Taking this guideline into account, the flow rates calculated above were multiplied by 2.5 in order to account for peak demand hours of the day. The resulting flow rates were then found to be 20.7 L/min, 15.7 L/min, and 5.0 L/min.

Design for a total water loss of 20-25% (leaks, valves left open, etc.)

Regardless of how well a system is designed, there will always be some water loss due to leaks from fittings and normal wear and tear. For our system, we will be assuming 20-25% leaks, which is reasonable based on the amount of leaks the existing system in Wasa has encountered. Taking leaks into account, the previously calculated flow rates are found to increase to 25.9 L/min, 19.6 L/min, and 6.3 L/min. These values are the minimum flow rates necessary in order to meet the demands of Nyamagola B.

The minimum capacity of each 'spigot' should be 10 liters/min. Each DP (distribution point) should be designed with a T having 2 spigots, so each DP should be able to provide 20 liters/min.

The minimum water capacity was already calculated in the above paragraphs, however, this guideline requires that the minimum flow rate be at least 20 liters/min. The number one priority was to provide the minimum amount of flowrate calculated above, while the second priority was to provide each distribution point with a minimum 20 L/min. For our design, we were able to provide very close to 20 L/min at each distribution point. Furthermore, each distribution point was still designed to have 2 spigots in order to help with demand during peak hours of the day.

The system should have a minimum water storage capacity equal to 50% of the average daily demand

While there are already a few water tanks in the existing system, the need for water storage is not a high priority in Wasa due to the nature of the above ground water source. Since the village does not have to worry about a pump failure, and the likelihood of the source going dry is extremely low, it does not make very good fiscal sense to direct budgetary costs toward water storage. Furthermore, in the unlikely event that the proposed water system would fail, the village would be able to revert back to the system they currently have as an emergency water source until repairs could be made.

One DP can serve a maximum of 250 people. Maximum walking distance to a DP is 400 m.
This design requirement was kept in the forefront while choosing the best possible distribution points. However, physical limitations did not always make this requirement a possibility. For example, because of the geographical limitations in Nyamagola B (i.e. sharp changes in elevation), it was not possible to extend the system further in order to provide an additional distribution point. Thus, the main distribution point in Nyamagola B was designed to serve 312 people rather than the maximum of 250 mentioned above. Since 312 people is not outrageously more than 250 people and the majority of them will not have to walk more than 400 m, it was deemed an acceptable design compromise given the circumstances and challenges faced.

The pipe surface roughness: PVC and HDPE 0.01 mm; galvanized steel 0.15 mm (relative roughness ε/d is roughness divided by internal pipe diameter)
Given that the Wasa system contains relatively low elevation changes, and thus, relatively low pressures in the pipes, most of the system will be modeled with PVC/HDPE at 0.01 mm. Galvanized pipe with a surface roughness of .15mm will be modeled for parts of the distribution points. In order to incorporate the surface roughness into the coefficient of friction calculations, the following equation was used:

$$\frac{1}{\sqrt{f}} = -1.8\log_{10}\left[\left(\frac{\varepsilon/d}{3.7}\right)^{1.11} + \frac{6.9}{\text{Re}}\right]$$

This equation was used under the assumption that the system was operating under turbulent flow, which is a fair assumption for a water distribution system. However, if the system was found to be laminar, the coefficient of friction would simply become f = 64/Re. During the calculations, an IF statement was used to calculate the friction factor based on the Reynolds number.
The maximum working pressure for a pipe should be approximately 80% of rating. For example: a HDPE pipe is rated at PN8. PN8 stands for 8 bars or 116 psig. Therefore, it shouldn't be used in environments where the pressure exceeds 0.8*116 psig, or 93 psig.
The maximum pressure occurs at the lowest elevation in the system when all of the valves are completely closed. The maximum pressure based on calculations for this system was used to calculate the pipe rating.

The velocity of water in a pipe should typically be in the range of 0.5 – 1.5 m/sec. Slower than 0.5 m/sec usually means the pipe is too large, though oversizing a gravity main is much preferable to under sizing it.
Each line in the system was sized based on the flow rate needed to deliver the downstream distribution point(s) with their adequate water demand. After sizing each line, the velocity in the

pipe was verified to be between 0.5 m/sec and 1.5 m/sec. This ensures that there are neither excessive frictional losses, nor excessive pipe sizes.

Lines should be buried a minimum of 1 meter. Sunlight degrades HDPE and farming practices can damage pipes laid near the surface

Since damages to the existing system were due in large part to a violation of this design requirement, the new system will emphasize this criteria during construction.

All minor losses should be modeled at 5% of major losses. Treat valves separately using Kv.

During our calculations of our system model, we multiplied the frictional coefficient by 1.05 in order to account for minor losses in the system. When completely opened, the valves at the distribution points were assumed to have a Kv value of 10. This was a standard assumption based on data from similar valves.

Add 15% to pipe costs for fittings; add 20% to supply costs (pipe/tank/concrete) for shipping.

These extra fittings and shipping cost were included in the cost analysis, which can be seen in detail under the section titled "Implementation Budget".

Proposed System

I. *Phase 1*

a. *Settling/cistern tank design*

A settling tank was designed to collect water from a spring source, settle out particles, and deliver clean water into the distribution pipes. The settling tank would consist of three parts, an inlet zone, a settling zone, and an outlet zone, which can be seen in Figure 16.

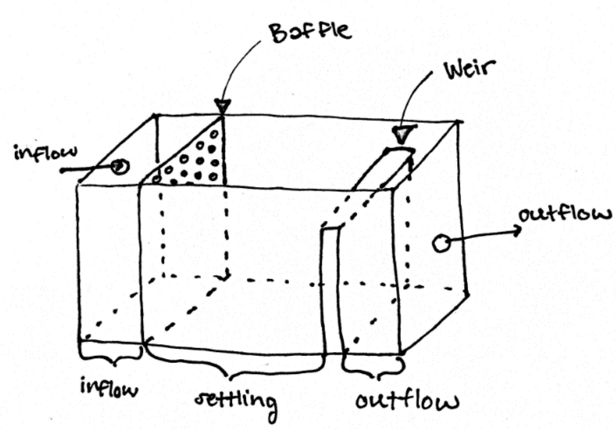

Figure 16: General sketch of cistern layout.

Design of the settling tank was dependent on two parameters; the capacity requirement of the system and settling velocity of the particles present at the source (coarse silt). The length dimension of the settling tank was dependent on the time needed for particles to settle, dimensions of the weir were dependent on desired capacity, and all other dimensions were dependent on shear and moment forces acting on the concrete.

The water would first enter the inlet zone through an inflow pipe and any excess water would be diverted to the existing cistern via an overflow pipe. The water would flow from the inlet zone through a baffle to smooth the flow into the settling zone. The length of the settling zone was calculated in terms of the particle size and a minimum length to width ratio of 4:1. After the water makes it through the settling zone and most of the particles are settled out, it would flow over a weir into the outlet zone. The weir smoothes out the flow of the water and acts as a final filter before it enters the distribution system. In the outlet zone the debris-free water would flow into the system to be distributed.

The cistern would also contain a shut off valve at the inlet and drains in the settling zone and outlet zone so that the tank could be cleaned periodically. The tank would be made out of reinforced concrete, the thickness of which was determined using the shear and moment equations for the water pressure acting on the concrete. The cistern design would be in accordance with the American Concrete Institute.

The proposed cistern would be placed adjacent to the existing cistern at the spring source for the main line and would have raised walls that are 0.30 meters above the ground to prevent water contamination due to runoff. It would also have a cover to prevent water contamination from animals and deter tampering.

The height of the inflow pipe is arbitrary but should be placed near the top of the tank so that it can fill as close to capacity as possible. The overflow pipe should be located just below the inflow pipe so that the water level stays below the inflow pipe and doesn't flow back out of the tank (a height of 1 meter is suggested for the overflow pipe). The outflow pipe should be located near the bottom of the outlet but not flush with the floor of the tank so that it doesn't get clogged with sediment but also doesn't intake any sediment that could still be floating at the surface of the outlet zone. The drains in the settling zone and outlet zone for cleaning should be placed slightly above the floor of the tank as well, to ensure that they drain a majority of the water but do not get clogged by sediment.

Steel reinforcement rods need to be placed both vertically and horizontally in the concrete to prevent cracks in the walls. Number 3 bars, which have a diameter of 0.375 inches are suggested and would be placed approximately 6.37 inches deep in the concrete and spaced 18 inches apart. More detail about the cistern design can be seen in Appendix C: Cistern Calculations.

b. Line to Nyamagola B

The other part of phase 1 involves getting water to Nyamagola as seen in table 2 and figure 17. A gravity main would be connected from the cistern (pt. 1) and would run southeast towards Nyamagola. A pipe would tee off at point 3 and supply the first distribution point (4) near the river on the southern edge of Nyamagola A. This distribution point would be placed in the middle of a group of houses approximately 100 meters north of the river. The other side of the tee at point 3 would continue south, across the river and into Nyamagola B. The second distribution point would be installed in Nyamagola B at point 5 to service the population center of Nyamagola B.

Table 2: Phase 1 critical locations.

Point	Description	Latitude	Longitude	Altitude (m)
1	Source - Settling tank	8° 6'42.84" S	35° 12'4.44" E	1768
2	Tee - Phase 2	8° 7'35.62" S	35° 12'26.20" E	1739
3	Tee - Distribution points	8° 7'41.91" S	35° 12'27.52" E	1724
4	Distribution point in Nyamagola A	8° 7'42.95" S	35° 12'26.42" E	1722
5	Distribution point in Nyamagola B	8° 7'55.04" S	35° 12'30.37" E	1744

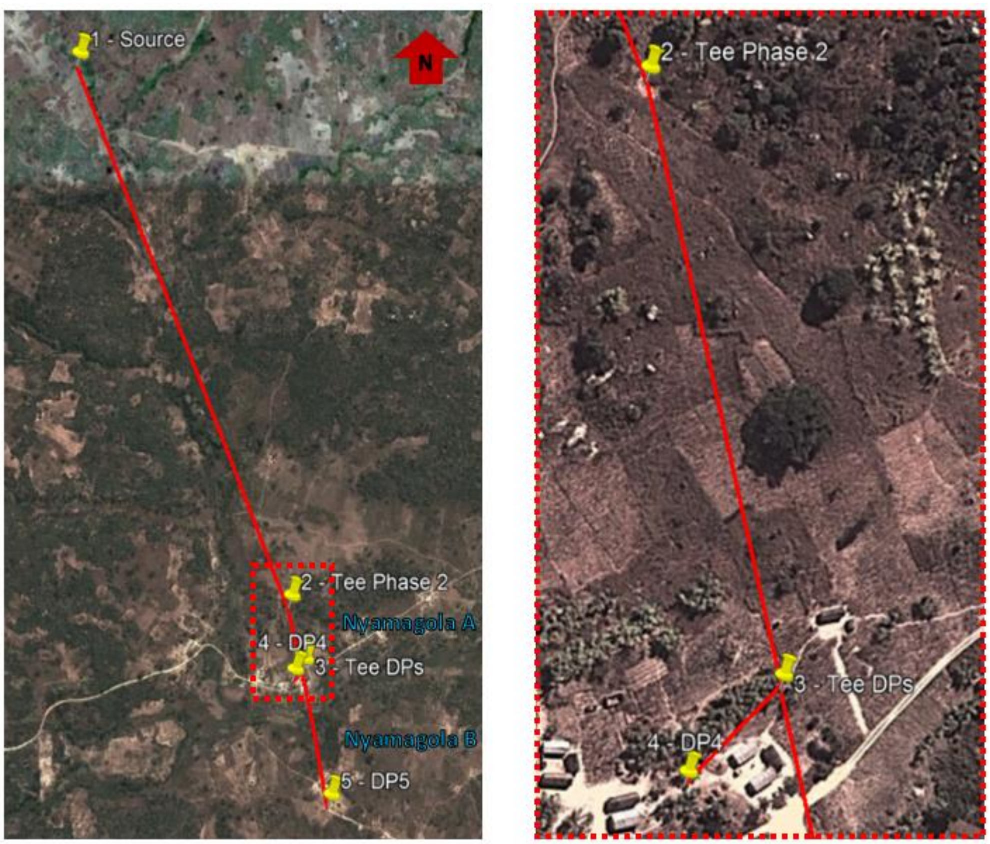

Figure 17: Phase 1 layout of system and distribution points in Nyamagola.

The proposed pipe lengths and sizes are described in table 3 below. The system was modeled in Engineering Equation Solver (EES), which can be seen in Appendix A. The maximum pressure calculated with all valves shut in the system was 72 psi. This pressure was located in the pipe across the Muhepasi river and will be handled by galvanized pipe. The rest of the system would see pressures lower than 72 psi and thus HDPE PE 100 PN 6 pipe could be used. The pipe from the source (1) to the phase 2 tee (2) would be 4in; the pipe from the phase 2 tee (2) to the distribution point tee (3) would be 1.25in. Both of these lines would be buried at least 1 meter deep, parallel to the existing main line. The trenching of the line is critical as it would cross under agricultural lands. The pipe from point 3 to distribution point 4 would be ½ in, the smallest realistic pipe size. This size produces more than enough water (see Appendix B), however any smaller pipe size might become more expensive due to availability issues. This section of the pipe would also pass through farmland and thus require proper trenching. The pipe from point 3 to distribution point 5 would be 1 in. This larger diameter is required due to the elevation changes from point 3 to 5 (see Appendix D). This section of pipe would travel south across the river to reach Nyamagola B. It would also be buried a meter deep until it reaches the river, where it would be elevated above the river to cross it. The pipe going over the river that would be exposed to sunlight and other potentially damaging factors would be galvanized steel rather than HDPE to avoid deterioration. Ball valves would be installed right before and after the river in case of emergencies. Both ends of the galvanized pipe would be anchored to the ground through concrete blocks. After the river, it would couple back to HDPE pipe and continue up a slight elevation incline and into Nyamagola B.

Table 3: Phase 1 pipe descriptions.

			Pipe Selection				
Pipe Location	Length (m)	Pressure Max (psi)	PE	PN	OD (mm)	WT (mm)	Nominal Bore (in)
1 ↔ 2	1770	43	100	6	110	4.2	4.00
2 ↔ 3	199	64	100	8	40	2.0	1.25
3 ↔ 4	47	67	100	8	20	1.2	0.50
3 ↔ 5	416	72	100	6	32	1.9	1.00

Ultimately, phase 1 would be able to provide ~43L/min to distribution point 4 and ~29L/min to distribution point 5. This is much more than the 6.3L/min needed at point 4 and the 19.6 L/min needed at point 5, however the system was designed with phase 2 in mind.

c. *Uhepwa Hand Pumps*

Uhepwa does not have access to either of Wasa's gravity fed systems. It is 1.5 kilometers south of Nyamagola B but is too far away and too high in elevation to supply water from the gravity systems in Nyamagola or Custom (see figure 6). Two wells would be drilled via mud rotary drilling and two hand-pump wells would be installed. This would supply most of the population of Uhepwa. One would be located to the east, near Uhepwa's existing water source (a spring replenished puddle at the base of a tree). Another hand pump would be installed to the west to cover the western part of Uhepwa. This hand pump would be installed near a center of houses. As mentioned above, the two hand pumps would be drilled via mud rotary. This method is expected to be successful as the underground springs in Uhepwa are not very deep (used as surface source) and provide water even during the dry season, indicating a relatively high water table. Furthermore, the geology of Uhepwa also makes it a good candidate for mud rotary drilling -- Figure 19 shows the landscape of Uhepwa -- which is relatively flat and moist since it lies at the bottom of a valley. The ground in Uhepwa is not rocky and the dirt is soft. Additional hand pumps could be considered in the future depending on population growth of Uhepwa and effectiveness of the Wasa water committee. Figure 18 below shows the approximate proposed locations of the two hand pumps in Uhepwa.

Figure 18: Uhepwa hand pump locations.

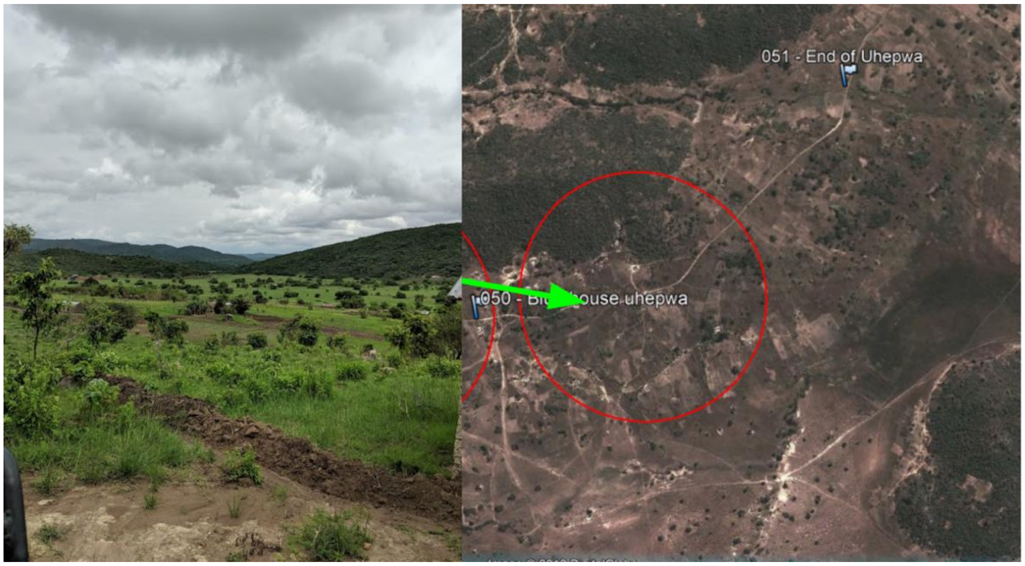

Figure 19: Uhepwa landscape .

II. Phase 2

a. Main line attachment

The second phase involves connecting the existing system to the system built in Phase 1. The current cistern would be shut off and the existing main line system would tap into the system proposed in Phase 1. This switch to the Phase 1 cistern would result in clean water since it would be raised and have a cover, protecting the spring water from contamination. The old system line from the source to the tee would be capped and left for emergency situations, or dug up and recycled. The old main line would tee into the new system in Nyamagola A right before it turns northwest, towards the source. This tee can be seen at the "2 - Tee Phase 2" in Figure 20 below. This phase would not require much labor or cost but does depend on the success of Phase 1. If the Wasa water committee can raise sufficient funds and properly maintain Phase 1, it is anticipated that Phase 2 would be implemented. This would provide the majority of Wasa with the safe, clean, and reliable water from Phase 1.

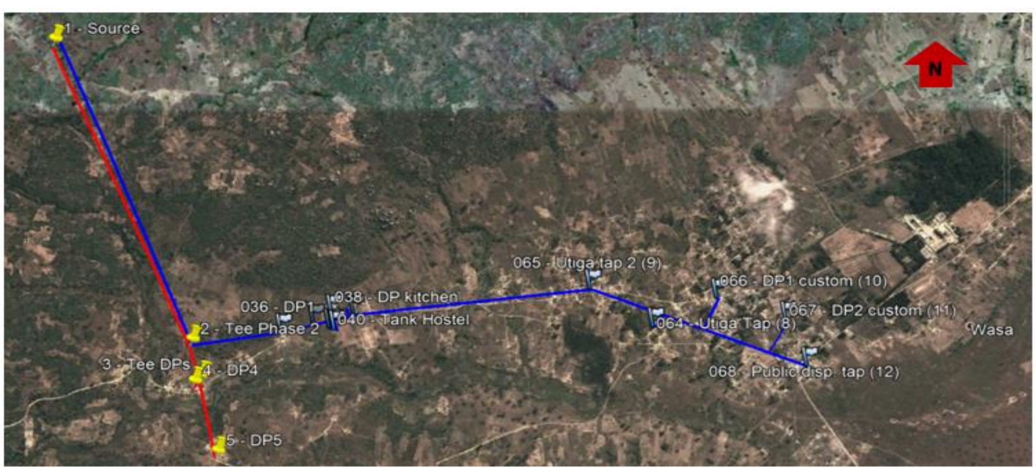

Figure 20: Map of the main line system (blue) and Phase 1 (red).

The ultimate goal of phase 2 directed the design of the system in phase 1. The flow measurements (as seen in figure 10) were added to give an approximate capacity of the existing system, approximately 215 L/min. The distance from point 2 (Tee Phase 2) to the closest distribution point is about 694 m and according to the fundi, 2.5in pipe was used. An imaginary distribution point was added to the system in the EES code to simulate the capacity of the existing system. The simulation in EES can be represented visually in figure 21 and numerically in tables 4 and 5. See Appendix A for more details.

Figure 21: Visual representation of system modeled in EES.

Table 4: Phase 2 critical locations.

Point	Description	Latitude	Longitude	Altitude (m)
1	Source - Settling tank	8° 6'42.84" S	35° 12'4.44" E	1768
2	Tee - Phase 2	8° 7'35.62" S	35° 12'26.20" E	1739
3	Tee - Distribution points	8° 7'41.91" S	35° 12'27.52" E	1724
4	Distribution point in Nyamagola A	8° 7'42.95" S	35° 12'26.42" E	1722
5	Distribution point in Nyamagola B	8° 7'55.04" S	35° 12'30.37" E	1744
6	Simulated existing system dist. pt.	8° 7'33.16" S	35° 12'40.82" E	1728

Table 5: Phase 2 pipe descriptions.

			Pipe Selection				
Pipe Location	Length (m)	Pressure Max	PE	PN	OD (mm)	WT (mm)	Nominal Bore (in)
1 ↔ 2	1770	43	100	6	110	4.2	4.00
2 ↔ 6	694	n/a	n/a	n/a	n/a	n/a	2.50
2 ↔ 3	199	64	100	8	40	2.0	1.25
3 ↔ 4	47	67	100	8	20	1.2	0.50
3 ↔ 5	416	72	100	6	32	1.9	1.00

Ultimately, Phase 2 would be able to provide 39 L/min to distribution point 4, 20 L/min to distribution point 5, and 296 L/min to the existing system. This is slightly more than what was needed (6.3L/min, 19.6L/min, and 215L/min, points 4, 5, and 6, respectively). Because the pipe running to distribution point 4 is already the smallest reasonable size, the only potential to cost savings would be to reduce the pipe sizes from the source (1) to distribution point 5 (5). Reducing either down to the next available pipe size was calculated and resulted in the system not being able to supply the required flow rates. Thus, the pipe sizes in table 5 show the optimal pipe size to provide Wasa enough water with minimized cost.

Impact of design

-Social Impact

The design for the proposed system is expected to reduce the occurrence of water-related diseases in Wasa as well as provide more people with access to not only water, but clean water. Many villagers go to rivers a great distance away to gather water, which is likely contaminated by agricultural runoff and rainwater. The newly designed system would allow for easier access to safe water because the cistern design would protect the source from agricultural runoff and contamination during the rainy season by having walls above the surface of the adjacent farmland, a cover, and an adequate settling zone to remove sediments in the water. Having a clean water source and system would reduce the number of people with water-related sickness and, in turn, reduce the burden on the medical dispensary to treat so many patients.

-Economic Impact

Economically, the proposed system would benefit Wasa by reducing the amount of time people in Nyamagola B spend getting water. The villagers could take advantage of this extra time by putting it towards economically beneficial activities like farming, education, and business. The reduction in waterborne illnesses would allow workers to spend more time doing their jobs instead of recovering from sickness.

Implementation Budget

An estimated budget of phase 1 is shown in table 6. It is broken into three cost categories; raw materials, in kind labor, and other. Availability and prices were estimated from DPI Simba Ltd (Tanzanian pipe supplier) catalogs and invoices/quotes from previous water projects. An exchange rate of USD 1 = TSH 2285 was used. As mentioned in the Phase 2 Design section, phase 2 cost is minimal and thus not shown.

Table 6 - Phase 1 budget.

Use	Description	Unit	Qty	Cost (USD)	Cost (TSH)	Total Cost (USD)
Dist. Pts.	Distirbution point	Each	2	176.57	403462.45	353.14
System fittings/pipe	HDPE Polypipe 4" PN6	150 meter	12	331.76	758071.60	3981.12
	HDPE Polypipe 1.25" PN8	150 meter	2	139.83	319511.55	279.66
	HDPE Polypipe 1/2" PN8	150 meter	1	44.00	100540.00	44.00
	HDPE Polypipe 1" PN6	150 meter	3	48.74	111370.90	146.22
	Pipe Fitings	USD	4804	15%	15%	720.62
Hand pump	Mud rotary well	Each	2	4200.00	9514470.00	8400.00
Cistern	Cenment	Bags	84	6.18	14121.30	519.12
	Reinforcing steel bar #3	Meter	93	2.50	5712.5	232.50
	Village supplied sand	Each	1	0.00	0	0.00
	Village supplied pebbles	Each	1	0.00	0	0.00
	Miscellaneous	EA	1	300.00	685500.00	300.00
Material Subtotal						**14976.38**
System fittings/pipe	Trench digging	Meter	2620	2.00	4570.00	5240.00
	Dist. Point Labor	Days	4	4.00	9140.00	16.00
Cistern	Cistern labor	Days	10	6.00	13710.00	60.00
In Kind Subtotal						**5316.00**
Cistern	Fundi	Ppl	1	300	6835500.00	300.00
	Sand source payment	Each	1	20.00	455700.00	20.00
	Pebble labor payment	Days	10	6.00	136710.00	60.00
	Village labor	Meter	120	2.00	45570.00	240.00
All	Transportation	Trips	2	176.00	4010160.00	352.00
Other Subtotal						**972.00**
Contingencies - 10% total						**2126.44**
Total						**23390.82**

Summary and Conclusions

The group's stay in Wasa provided vital information about the way of life of the Wasa residents, as well as their water distribution needs. Currently, there are two water distribution systems in Wasa. The "Mission Line" services the Catholic Church and Mission, the primary school, and the subvillage of Itawi, while the "Main Line" services the secondary school, a medical dispensary and the sub-villages of Custom, Utiga, Nyamagola A, and part of Itawi. Both systems are fed by spring sources, but are contaminated due to poor cistern designs that allow rain water run off to flow into them.

Meeting with the water committee revealed two priority areas, both of which the committee members felt were important to be reached with clean water. The first priority area is the small, isolated sub-village of Uhepwa. Due to distance and elevation challenges, it was quickly realized that neither the Main Line nor the Mission Line would be able to deliver water to Uhepwa. However, it is believed that mud rotary drilling could be effective in Uhepwa, thus, Phase 1 of the design includes two hand pumps for the sub-village. The second priority area is the sub-village of Nyamagola B. Located on the south side of a small river across from Nyamagola A, this sub-village and its 52 households currently do not have access to clean water. Additionally in Phase 1, the design includes creating a settling tank at the Main Line source and running a gravity main parallel to the existing system to Nyamagola B. During Phase 2, the Main Line cistern and gravity main will be disconnected and the existing taps on the Main Line will be connected to the new gravity main from the Nyamagola B system.

In order to design a new, effective system, the Tanzanian Design Guidelines were followed. These requirements provided the ability to calculate the water demands in Nyamagola B, taking into account population growth over the next ten years. Due to our villages unique circumstances and needs, it is worth noting that not every requirement in the guidelines were followed, however, a full breakdown of each design requirement can be found in the Tanzanian Design Guidelines section of the report. Next, the new system was modeled using principles of Fluid Mechanics and the complex system of equations was solved simultaneously using the computer software, EES. After an acceptable model was created, cost calculations were performed for the new system, the new settling tank, and the two hand pumps. The cost of each was calculated to be $5,289, $1,453, and $8,400 , respectively. Additionally, a 10% contingency budget of $2081 is included. Thus, bringing clean water to the aforementioned sub-villages of Wasa will cost roughly $22,891 total, $5,316 of which is expected to be sourced from in-kind contribution.

References

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Appendix A - EES Code

```
{--------------------------------WASA---------------------------------------}
{For other variables, see parametric table}
Function ff(Re, ed)
If (Re>2300) Then
	ff:=1.05/(1.8*log10((ed/3.7)^1.11 + 6.9/Re))^2	{1.05 factor to account for minor losses}
Else
	ff:=1.05*64/Re	{1.05 factor to account for minor losses}
Endif
End


{CONSTANTS}
rho=1000{kh/m^3} {density of water}
mu=.0011	{Ns/m^2} {dynmaic viscosity of water}
epsilonH=.01/1000 {m} {inner pipe wall roughness for HDPE}
epsilonG=.15/1000 {m} {inner pipe wall roughness for galvanized}
g=9.81	{m/s^2}



{---------------------PHASE 1-----------------------}
{
{Pipe lengths, m}
L12= 1770	{source to phase 2 tee}
L23= 199	{phase 2 tee to dist. pts. tee}
L34= 47	{dist. pts. tee to DP4}
L4dp=2	{DP4 accounting for vertical pipe rise to valves}
L35= 416	{dist. pts. tee to DP4}
L5dp=2	{DP5 accounting for vertical pipe rise to valves}

{DIAMETER, m}
d12= .1016	{PE100 PN6 OD110 WT4.2 (4")}
d23= .0360	{PE100 PN8 OD40 WT2 (1.25")}
d34= .0176	{PE100 PN8 OD20 WT1.2 (1/2")}
d4dp=.0176	{PE100 PN8 OD20 WT1.2 (1/2")}
d35= .0282	{PE100 PN6 OD32 WT1.9 (1")}
d5dp=.0282	{PE100 PN6 OD32 WT1.9 (1")}

{RELATIVE ROUGHNESS}
ed12=epsilonH/d12
ed23=epsilonH/d23
ed34=epsilonH/d34
ed4dp=epsilonG/d4dp
ed35=epsilonH/d35{galvanized section across river not modeled, assumed negligible effect}
ed5dp=epsilonG/d5dp

{AREA OF PIPE XSECT, INTERNAL, m}
A12=pi*d12^2/4
```

```
A23=pi*d23^2/4
A34=pi*d34^2/4
A4dp=pi*d4dp^2/4
A35=pi*d35^2/4
A5dp=pi*d5dp^2/4

{KNOWN ELEVATIONS, m}
z1= 1768
z2= 1739-1        {-1 accounting for 1 meter deep trenching}
z3= 1724-1        {-1 accounting for 1 meter deep trenching}
z4= 1722-1        {-1 accounting for 1 meter deep trenching}
z4dp=z4+2         {+2 accounting for vertical rise from trench to 1 meter above concrete pad}
z5= 1744-1        {-1 accounting for 1 meter deep trenching}
z5dp=z5+2         {+2 accounting for vertical rise from trench to 1 meter above concrete pad}

{VOLUMETRIC FLOW RATES}
Q12=Q23
Q23=Q34+Q35
Q34=Q4dp
Q35=Q5dp

{AVERAGE VELOCITIES IN PIPE SECTIONS, m/s}
V12=Q12/A12
V23=Q23/A23
V34=Q34/A34
V4dp=Q4dp/A4dp
V35=Q35/A35
V5dp=Q5dp/A5dp

{REYNOLDS NUMBERS}
Re12=rho*V12*d12/mu
Re23=rho*V23*d23/mu
Re34=rho*V34*d34/mu
Re4dp=rho*V4dp*d4dp/mu
Re35=rho*V35*d35/mu
Re5dp=rho*V5dp*d5dp/mu

{CALL FRICTION FACTOR ROUTINE}
f12=ff(Re12, ed12)
f23=ff(Re23, ed23)
f34=ff(Re34, ed34)
f4dp=ff(Re4dp, ed4dp)
f35=ff(Re35, ed35)
f5dp=ff(Re5dp, ed5dp)

{BOUNDARY CONDITIONS}
P1=0
P4dp=0
P5dp=0

{GOVERNING EQUATIONS}
(P2-P1) / (rho * g) + z2 - z1 = -(V12^2) / (2 * g) * ( f12 * L12 / d12)
(P3-P2) / (rho * g) + z3 - z2 = -(V23^2) / (2 * g) * ( f23 * L23 / d23)
(P4-P3) / (rho * g) + z4 - z3 = -(V34^2) / (2 * g) * ( f34 * L34 / d34)
```

```
(P4dp-P4) / (rho * g) + z4dp - z4 = -(V4dp^2) / (2 * g) * ( f4dp * L4dp / d4dp + K4dp)
(P5-P3) / (rho * g) + z5 - z3 = -(V35^2) / (2 * g) * ( f35 * L35 / d35)
(P5dp-P5) / (rho * g) + z5dp - z5 = -(V5dp^2) / (2 * g) * ( f5dp * L5dp / d5dp + K5dp)

{CONVERT}
P2psig=P2*14.7/101325
P3psig=P3*14.7/101325
P4psig=P4*14.7/101325
P5psig=P5*14.7/101325
Q12Lm=Q12*1000*60   {l/min}
Q23Lm=Q23*1000*60   {l/min}
Q34Lm=Q34*1000*60   {l/min}
Q4dpLm=Q4dp*1000*60   {l/min}
Q35Lm=Q35*1000*60   {l/min}
Q5dpLm=Q5dp*1000*60   {l/min}
 }

{----------------------PHASE 2------------------------}

{Pipe lengths, m}
L12= 1770            {source to phase 2 tee}
L23= 199 {phase 2 tee to dist. pts. tee}
L34= 47   {dist.pts. tee to DP4}
L4dp=2   {DP4 accounting for vertical pipe rise to vavles}
L35= 416 {dist.pts. tee to DP5}
L5dp=2   {DP5 accounting for vertical pipe rise to vavles}
L26= 694

{DIAMETER, m}
d12= .1016          {PE100 PN6 OD110 WT4.2 (4")}
d23= .0360          {PE100 PN8 OD40 WT2 (1.25")}
d34= .0176          {PE100 PN8 OD20 WT1.2 (1/2")}
d4dp=.0176          {PE100 PN8 OD20 WT1.2 (1/2")}
d35= .0282          {PE100 PN6 OD32 WT1.9 (1")}
d5dp=.0282          {PE100 PN6 OD32 WT1.9 (1")}
d26= .0625          {Existing pipe 2.5"}

{RELATIVE ROUGHNESS}
ed12=epsilonH/d12
ed23=epsilonH/d23
ed34=epsilonH/d34
ed4dp=epsilonG/d4dp
ed35=epsilonH/d35
ed5dp=epsilonG/d5dp
ed26=epsilonH/d26 {Existing pipe HDPE}

{AREA OF PIPE XSECT, INTERNAL, m}
A12=pi*d12^2/4
A23=pi*d23^2/4
A34=pi*d34^2/4
A4dp=pi*d4dp^2/4
A35=pi*d35^2/4
A5dp=pi*d5dp^2/4
A26=pi*d26^2/4
```

```
{KNOWN ELEVATIONS, m}
z1= 1768
z2= 1739-1          {-1 accounting for 1 meter deep trenching}
z3= 1724-1          {-1 accounting for 1 meter deep trenching}
z4= 1722-1          {-1 accounting for 1 meter deep trenching}
z4dp=z4+2           {+2 accounting for vertical rise from trench to 1 meter above concrete pad}
z5= 1744-1          {-1 accounting for 1 meter deep trenching}
z5dp=z5+2           {+2 accounting for vertical rise from trench to 1 meter above concrete pad}
z6= 1728

{VOLUMETRIC FLOW RATES}
Q12=Q23+Q26
Q23=Q34+Q35
Q34=Q4dp
Q35=Q5dp

{AVERAGE VELOCITIES IN PIPE SECTIONS, m/s}
V12=Q12/A12
V23=Q23/A23
V34=Q34/A34
V4dp=Q4dp/A4dp
V35=Q35/A35
V5dp=Q5dp/A5dp
V26=Q26/A26

{REYNOLDS NUMBERS}
Re12=rho*V12*d12/mu
Re23=rho*V23*d23/mu
Re34=rho*V34*d34/mu
Re4dp=rho*V4dp*d4dp/mu
Re35=rho*V35*d35/mu
Re5dp=rho*V5dp*d5dp/mu
Re26=rho*V26*d26/mu

{CALL FRICTION FACTOR ROUTINE}
f12=ff(Re12, ed12)
f23=ff(Re23, ed23)
f34=ff(Re34, ed34)
f4dp=ff(Re4dp, ed4dp)
f35=ff(Re35, ed35)
f5dp=ff(Re5dp, ed5dp)
f26=ff(Re26, ed26)

{BOUNDARY CONDITIONS}
P1=0
P4dp=0
P5dp=0
P6=0

{GOVERNING EQUATIONS}
(P2-P1) / (rho * g) + z2 - z1 = -V12^2 / (2 * g) * (f12 * L12 / d12)
(P3-P2) / (rho * g) + z3 - z2 = -V23^2 / (2 * g) * ( f23 * L23 / d23)
(P4-P3) / (rho * g) + z4 - z3 = -V34^2 / (2 * g) * ( f34 * L34 / d34)
```

```
(P4dp-P4) / (rho * g) + z4dp - z4 = -V4dp^2 / (2 * g) * ( f4dp * L4dp / d4dp + K4dp)
(P5-P3) / (rho * g) + z5 - z3 = -V35^2 / (2 * g) * ( f35 * L35 / d35)
(P5dp-P5) / (rho * g) + z5dp - z5 = -V5dp^2 / (2 * g) * ( f5dp * L5dp / d5dp + K5dp)
(P6-P2) / (rho * g) + z6 - z2 = -V26^2 / (2 * g) * ( f26 * L26 / d26 + K26)


{CONVERT}
P2psig=P2*14.7/101325
P3psig=P3*14.7/101325
P4psig=P4*14.7/101325
P5psig=P5*14.7/1010325
Q12Lm=Q12*1000*60   {l/min}
Q23Lm=Q23*1000*60   {l/min}
Q34Lm=Q34*1000*60   {l/min}
Q35Lm=Q35*1000*60   {l/min}
Q4dpLm=Q4dp*1000*60   {l/min}
Q5dpLm=Q5dp*1000*60   {l/min}
Q26Lm=Q26*1000*60 {l/min}
```

Appendix B - Water System Performance/EES Output

Parametric Table: PHASE 1

	K4dp	K5dp	Q12Lm	Q23Lm	Q34Lm	Q35Lm	P2psig	P3psig	P4psig
Run 1	10	10	72.43	72.43	43.3	29.13	41.86	49.78	12.07
Run 2	1.000E+10	1.000E+10	0.0123	0.0123	0.004337	0.007961	42.7	64.04	66.89

Parametric Table: PHASE 1

	P5psig	V12	V23	V34	V35	V4dp	V5dp
Run 1	3.398	0.1489	1.186	2.967	0.7773	2.967	0.7773
Run 2	35.58	0.00002528	0.0002014	0.0002971	0.0002124	0.0002971	0.0002124

Parametric Table: PHASE 2

	K26	K4dp	K5dp	Q12Lm	Q23Lm	Q26Lm	Q34Lm	Q35Lm	P2psig
Run 1	10	10	10	354.7	58.96	295.7	38.93	20.03	28.78
Run 2	1.000E+10	1.000E+10	1.000E+10	0.06387	0.0123	0.05157	0.004337	0.007961	42.7

Parametric Table: PHASE 2

	P3psig	P4psig	P5psig	V12	V23	V26	V34	V35
Run 1	40.84	10.31	0.3119	0.7291	0.9654	1.606	2.667	0.5345
Run 2	64.04	66.89	3.568	0.0001313	0.0002014	0.0002801	0.0002971	0.0002124

Figure B.1: System performance/EES output. K [=] unitless, Q [=] L/min, P [=] psi, V [=] m/s

Appendix C – Cistern Calculations

This appendix contains information on sizing the cistern to achieve sediment of particles and desired flow rate, followed by structural calculations to design the reinforced concrete cistern.

Sizing the Cistern:

The plan area of the settling tank was determined using the following equation from (Erosion and Sediment Control Manual: Appendix G, 2011) with the settling velocity determined from Table C.1 assuming coarse silt at 20 ^oC:

$$A_{plan} = 1.2 * (\frac{Q}{V_s}) \quad (1)$$

A_{plan} = Plan Area of Settling Tank (m^2)

Q = Desired Outflow Rate (From EES code Q = 381 L/min. = 6.35×10^{-3} m^3/sec)

V_s = Settling Velocity (m/s) (From Table C.1, assumed particle size of 0.04 mm at 20 °C)

This resulted in A_{plan} of 5.48 m^2.

Table C.1 Particle Settling Velocities (Coarse Silt at 20 ^oC used) (Erosion and Sediment Control Manual: Appendix G, 2011)

Diameter	Settling Velocity in Centimeters per Second					
(MM)	0 ℃	5 ℃	10 ℃	15 ℃	20 ℃	Particle
0.01	0.005	0.006	0.007	0.008	0.009	Fine Silt
0.02	0.020	0.023	0.027	0.031	0.035	Median Silt
0.03	0.044	0.052	0.060	0.069	0.078	
0.04	0.078	0.092	0.107	0.122	0.139	Coarse Silt
0.05	0.122	0.143	0.167	0.191	0.217	
0.06	0.176	0.207	0.240	0.275	0.313	
0.07	0.239	0.281	0.327	0.375	0.426	Very Fine Sand
0.08	0.312	0.367	0.427	0.490	0.556	
0.09	0.395	0.465	0.540	0.620	0.704	
0.110	0.488	0.574	0.667	0.765	0.869	
0.11	0.590	0.694	0.807	0.926	1.051	
0.12	0.703	0.826	0.960	1.101	1.251	
0.13	0.825	0.970	1.127	1.293	1.468	Fine Sand
0.14	0.956	1.125	1.307	1.499	1.703	
0.15	1.098	1.291	1.501	1.721	1.955	
0.16	1.249	1.469	1.707	1.958	2.224	
0.17	1.410	1.658	1.928	2.211	2.511	
0.18	1.581	1.859	2.161	2.478	2.815	
0.19	1.761	2.072	2.408	2.761	3.136	
0.20	1.952	2.295	2.668	3.060	3.475	
℉	32	41	50	59	68	

The dimensions for sizing the settling zone of the cistern were determined from Table C.2 and included the ratio of the length of the settling zone to the width of the settling zone (i.e., 4:1) and length of the settling zone to the depth of the cistern (i.e., 15:1).

Table C.2 Sedimentation basin design guidelines for horizontal-flow rectangular sedimentation basins (Davis, 2010).

Parameter	Typical Range of Values	Comment
Inlet Zone		
Distance to Diffuser Wall	2 m	
Diffuser Hole Diameter	0.10 – 0.20 m	
Settling Zone		
Overflow Rate	40 – 70 $m^3/d*m^2$	
Side Water Depth (SWD)	3 – 5 m	
Length	30 m	Wind Constraint
	60 m	Chain-and-flight
	≥ 80 – 90 m	Traveling Bridge
Width	0.3 m increments	Chain-and-flight
	6 m maximum per train	Chain-and-flight
	24 m maximum = 3 trains/drive	Chain-and-flight
	30 m maximum	Traveling Bridge
L:W	4:1 to 6:1	≥6:1 preferred
L:D	15:1	Minimum
Velocity	0.005 – 0.018 m/s	Horizontal, mean
Reynolds Number	< 20,000	
Froude Number	> 10^{-5}	
Outlet Zone		
Launder Length	1/3 – 1/2 length of basin	Evenly spaced
Launder Weir Loading	140 – 320 $m^3/d*m$	
Sludge Zone		
Depth	0.6 – 1 m	Equipment dependent
Slope	1:600	Mechanical cleaning
Sludge Collector Speed	0.3 – 0.9 m/min.	

Based on the ratio of the length (L):width (b) of 4 and knowing A_{plan} of 5.48 m^2, the length and width of the cistern were found to be $L*b=(4b*b)=A_{plan}= 5.48\ m^2 \rightarrow b= 1.17$ m and $L = 4.68$m Because these guidelines are intended for full-scale water treatment plants, the 15:1 ratio for length:depth would have led to an unreasonably long cistern. It is not critical that this design requirement is met because it is a guideline for designing U.S. wastewater treatment plants but does not have an impact on the overall settling rate of particles. For settling of particles, the area is the most important parameter (Hozalski 2018).

Table C.2 also suggests that the length of the inlet zone and the length of the outlet zone each be approximately 1/3 the length of the settling zone of the cistern. These recommendations were followed with each zone being 1/3 of the length. The inlet zone, settling zone, and outlet zone can be seen in Figure C.1

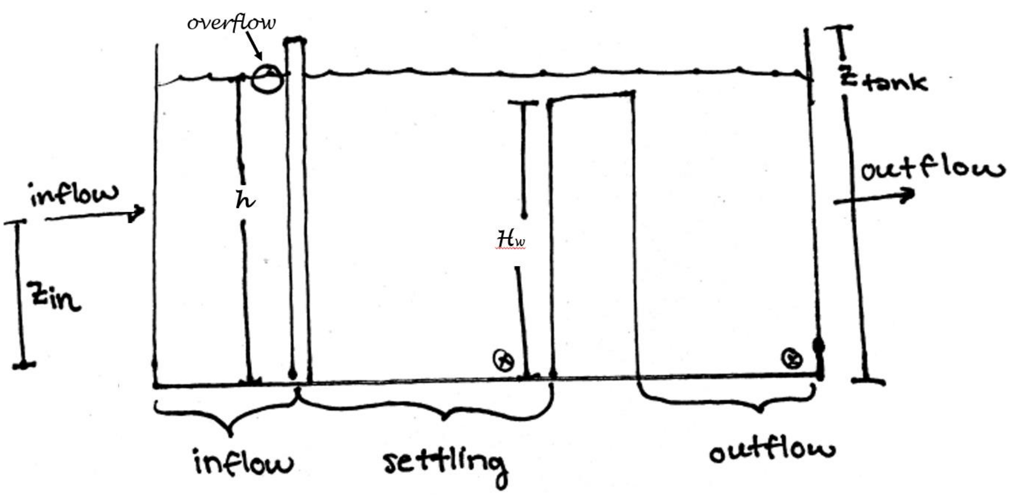

Figure C.1 Elevation view of cistern showing inlet/inflow zone, settling zone, outlet/outflow zone, water depth (*h*),weir height (H_w), and tank height (*z*) (not to scale)

Sizing the Weir:

Equations 2, 3, and 4 are from "Sharp-Crested Weir Discharge Coefficient." (Arvanaghi 2013). It was assumed for these calculations that the depth of the water was 1 m. This value was chosen arbitrarily but was assumed to be reasonable. Knowing *h* and the desired outflow rate (*Q*), the height of the weir (H_w) could be determined.

$$Q = C_w * b * (h - H_w)^{\frac{3}{2}} \qquad (2)$$

$$C_w = \frac{2}{3} * C_d * \sqrt{2g} \qquad (3)$$

$$C_d = 0.611 + 0.08\frac{h}{H_w} \qquad (4)$$

Combining equations 2, 3, and 4:

$$Q = \frac{2}{3}\left(0.611 + 0.08\frac{H}{H_w}\right)\sqrt{2g} * b(H - H_w)^{\frac{3}{2}} \qquad (5)$$

Q = Desired Outflow Rate (From EES code Q = 381 L/min. = 6.35 x 10^{-3} m^3/sec)
C_w = Weir Coefficient
b = Width of Cistern (1.17 m)
h = Depth of Water (m); assumed to be 1 m
H_w = Height of Weir (m)
C_d = Discharge Coefficient
g = Acceleration due to Gravity (9.8 m/s^2)

The height of the weir, H_w, was determined by plugging values into equation 5 and using a quadratic equation solver.

<u>Structural Design of Cistern:</u>

<u>Required Cistern Wall Thickness to Resist Shear:</u>

The design for shear was based on using ACI 318-14 structural concrete building code requirements. The design for shear requires the capacity of the member to exceed the demand or factored shear (i.e., $\phi V_n \geq \gamma V_{expected}$ or V_u). The capacity of the cistern wall in shear is the nominal shear resistance of the wall at failure (V_n) multiplied by an undercapcity factor, ϕ_{shear}, to account for uncertainties such as variations in material properties, ability of the model to predict behavior, etc. The required shear or shear demand is based on the shear expected to be carried by the member under everyday loading conditions multiplied by a load factor, LF, to amplify the expected load to what would be expected to produce failure in the member. The equations for the shear capacity and demand per unit length of the cistern wall are provided in Equations 6 and 7, respectively. The everyday loading conditions in Equation 6 are created by the load generated due to the unit weight of water. The load factor, LF, to amplify the loads to imminent failure, is taken as 1.4. This factor is suggested by ACI 318-14 for fluid loads. In equation 7, the undercapacity factor, ϕ, is taken as 0.75. The model used to provide the resistance of the section to shear is based on the tensile resistance of concrete to shear cracking.

$$\frac{V_{demand}}{L} = \frac{1}{2}(\gamma h)h*(1.4) \tag{6}$$

$$\frac{V_{capacity}}{L} = \phi_{\text{shear}} * \ 2\sqrt{f'_c} * \ d \tag{7}$$

V_{demand} = Shear Force due to water pressure on cistern wall (lb)

$V_{capacity}$ = Shear Resistance of the concrete (lb)

L = Unit Length of Cistern (1 ft per ft of length)

γ = Unit Weight of Water (62.4 pcf)

h = Depth of Water (ft)

f_c' = Compressive Strength of Concrete (2500 psi)

d = Distance to Centroid of Reinforcement from extreme compression face through member thickness (in)

ϕ_{shear} = Undercapacity Factor for Shear (0.75)

Requiring the capacity to exceed the demand results in the determination of a minimum effective depth, d. The minimum thickness of the wall is then equal to d + bar radius + 3 in., where the 3 in. reflects the required concrete cover to protect the reinforcement.

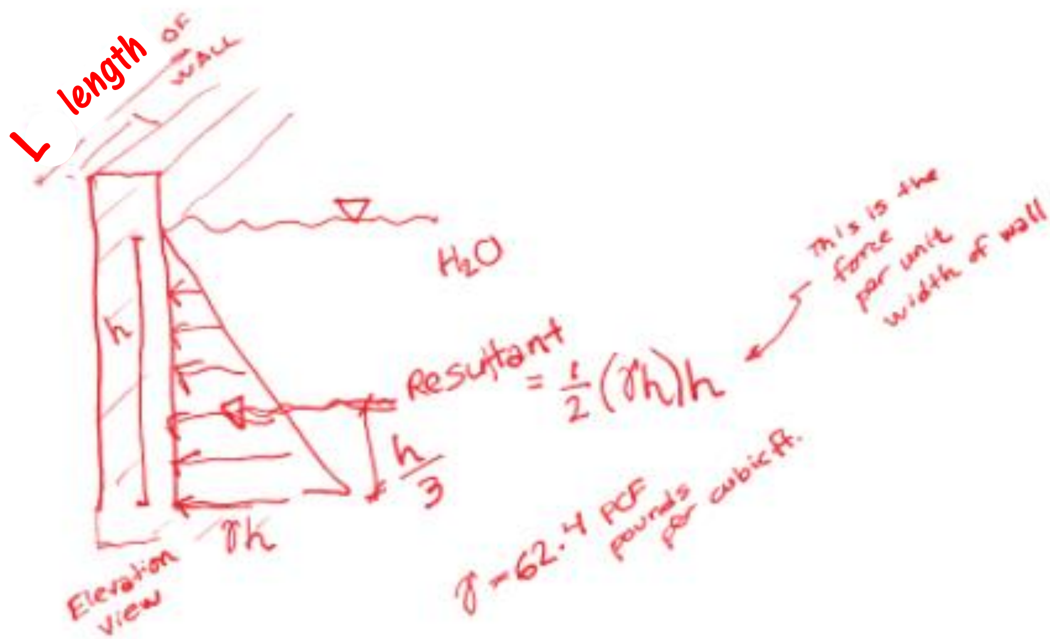

Figure C.2 A sketch of variables used to calculate shear stress on concrete (Cathy French, 2018).

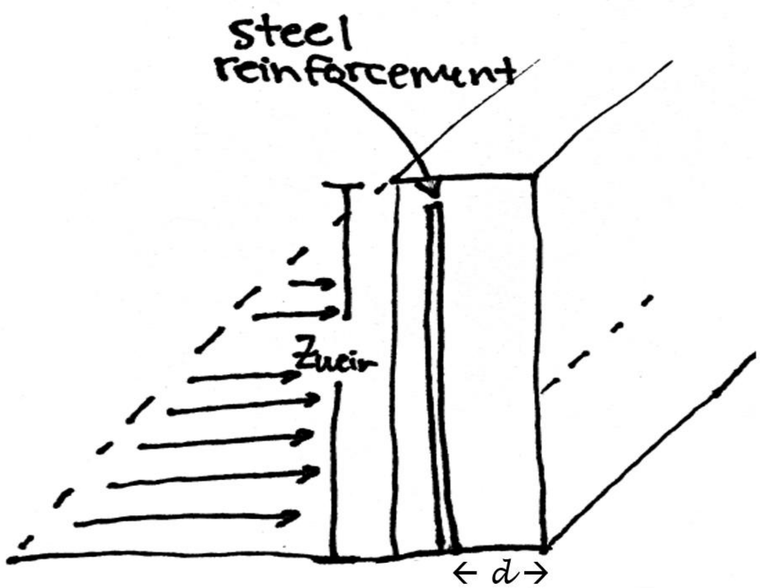

Figure C.3 A sketch showing the physical interpretation of "*d*," the distance from the extreme compressive face in the concrete to the centroid of the flexural reinforcement.

Required Area of Reinforcement to Resist Flexure:

Similar to the design for shear, the design for flexure was based on using ACI 318-14 structural concrete building code requirements. The design for flexure requires the capacity of the member to exceed the demand or factored moment (i.e., $\phi M_n \geq \gamma M_{expected}$ or M_u). The capacity of the cistern wall in flexure is the nominal flexural resistance of the wall at failure (M_n) multiplied by an undercapcity factor, ϕ_{flexure}, to account for uncertainties such as variations in material properties, ability of the model to predict behavior, etc. The required flexural demand is based on the moment expected to be carried by the member under everyday loading conditions multiplied by a load factor, LF, to amplify the expected load to what would be expected to produce failure in the member. The equations for the flexural capacity and demand per unit length of the cistern wall are provided in Equations 8 and 9, respectively. The everyday loading conditions in Equation 8 are created by the load generated due to the unit weight of water multiplied by the moment arm of the resultant force, *h*/3. The load factor, LF, to amplify the loads to imminent failure, is taken as 1.4, which was the factor suggested by ACI 318-14 for fluid loads. In equation 9, the undercapacity factor for flexure, ϕ_{flexure}, is taken as 0.9. The model used to provide the resistance of the section to flexure assumes the concrete crushes at failure and the reinforcement yields in tension.

Moment per unit length of wall:

$$\frac{M_{demand}}{L} = \frac{1}{3}h * \left[\frac{1}{2}(\gamma h)h * (1.4)\right] \quad (8)$$

$$\frac{M_{capacity}}{L} = \emptyset_{flexure} A_s f_y \left[d - \frac{A_s f_y}{2*0.85*L*f_c'}\right] \quad (9)$$

M = Moment Demand on Concrete per Unit Length (lb*ft/per ft of length of wall)
L = Unit Length of Wall (12in./ft of length of wall)
h = Height of Water (ft)
ϕ_{flexure} = Undercapacity Factor for Flexure (0.90)
A_s = Area of Steel Reinforcement per Unit Length (in^2/ft of length of wall)
f_y = Yield Strength of Steel (60,000 psi)
d = Effective Depth of Reinforcement from Extreme Compression Face of Wall (in)
f_c' = Compressive Strength of Concrete (2500 psi)

Solve equation 8 for "M_{demand}" and then plug "M_{demand}" into "$M_{capacity}$" in equation 9 along with the rest of the known values. Using a quadratic equation solver, solve for A_s and use Table C.3 to determine how many #3 bars per ft of wall should be used.

Table C.3 Table used to calculate number of reinforcement bars needed (#3 bars assumed). With an area of reinforcement of 0.0168 in^2/ft, the maximum spacing distance is recommended (18 in.)

Areas of Bars in a Section 1 ft wide (in^2/ft) REVISED

Bar Spacing (in.)	No. 3	No. 4	No. 5	No. 6	No. 7
4	0.33	0.60	0.93	1.32	1.80
4.5	0.29	0.53	0.83	1.17	1.60
5	0.26	0.48	0.74	1.06	1.44
5.5	0.24	0.44	0.68	0.96	1.31
6	0.22	0.40	0.62	0.88	1.20
6.5	0.20	0.37	0.57	0.81	1.11
7	0.19	0.34	0.53	0.75	1.03
7.5	0.18	0.32	0.50	0.70	0.96
8	0.17	0.30	0.47	0.66	0.90
8.5	0.16	0.28	0.44	0.62	0.85
9	0.15	0.27	0.41	0.59	0.80
9.5	0.14	0.25	0.39	0.56	0.76
10	0.13	0.24	0.37	0.53	0.72
10.5	0.13	0.23	0.35	0.50	0.69
11	0.12	0.22	0.34	0.48	0.65
11.5	0.11	0.21	0.32	0.46	0.63
12	0.11	0.20	0.31	0.44	0.60
13	0.10	0.18	0.29	0.41	0.55
14	0.09	0.17	0.27	0.38	0.51
15	0.09	0.16	0.25	0.35	0.48
16	0.08	0.15	0.23	0.33	0.45
17	0.08	0.14	0.22	0.31	0.42
18	0.07	0.13	0.21	0.29	0.40

The reinforcement bars must bend into the base of the concrete tank so that they are anchored to develop the required forces. The distance they should continue after the bend is given by the following equation from ACI 318-14 Chapter 25:

$$l_d = \left(\frac{f_y}{50*\sqrt{f_c'}}\right) * d_b \qquad (10)$$

l_d = Development Length of Bar After Bend (in)
f_y = Yield Strength of Steel (60,000 psi)
f_c' = Compressive Strength of Concrete (2500 psi)
d_b = Diameter of Bar (0.375 in)

The flexural reinforcement must be placed vertically to resist the moment. In addition, horizontal reinforcement is required for shrinkage and temperature to control potential cracking in the concrete. The maximum spacing of the reinforcement is 18 in. or 0.45 meters. Sketches of the reinforcement are shown in Figure C.4 and Figure C.5.

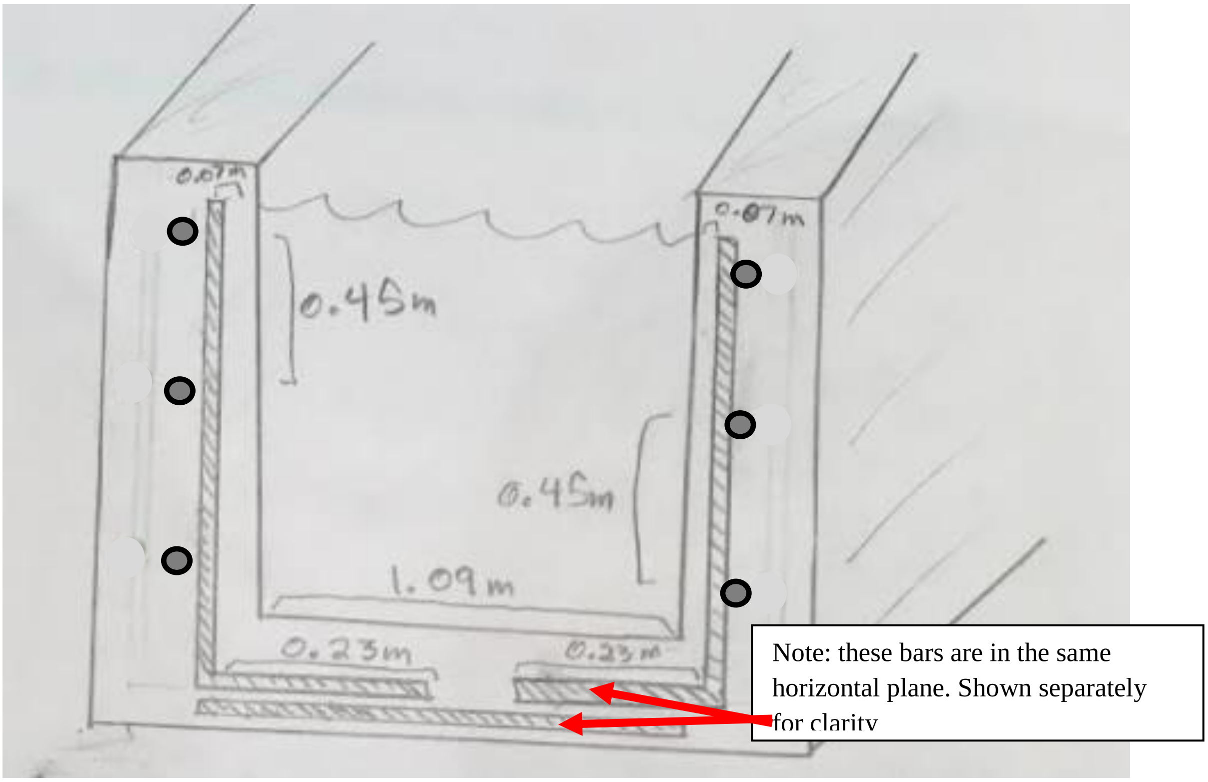

Figure C.4 A sketch of the cross section of the cistern showing the placement of the reinforcement bars.

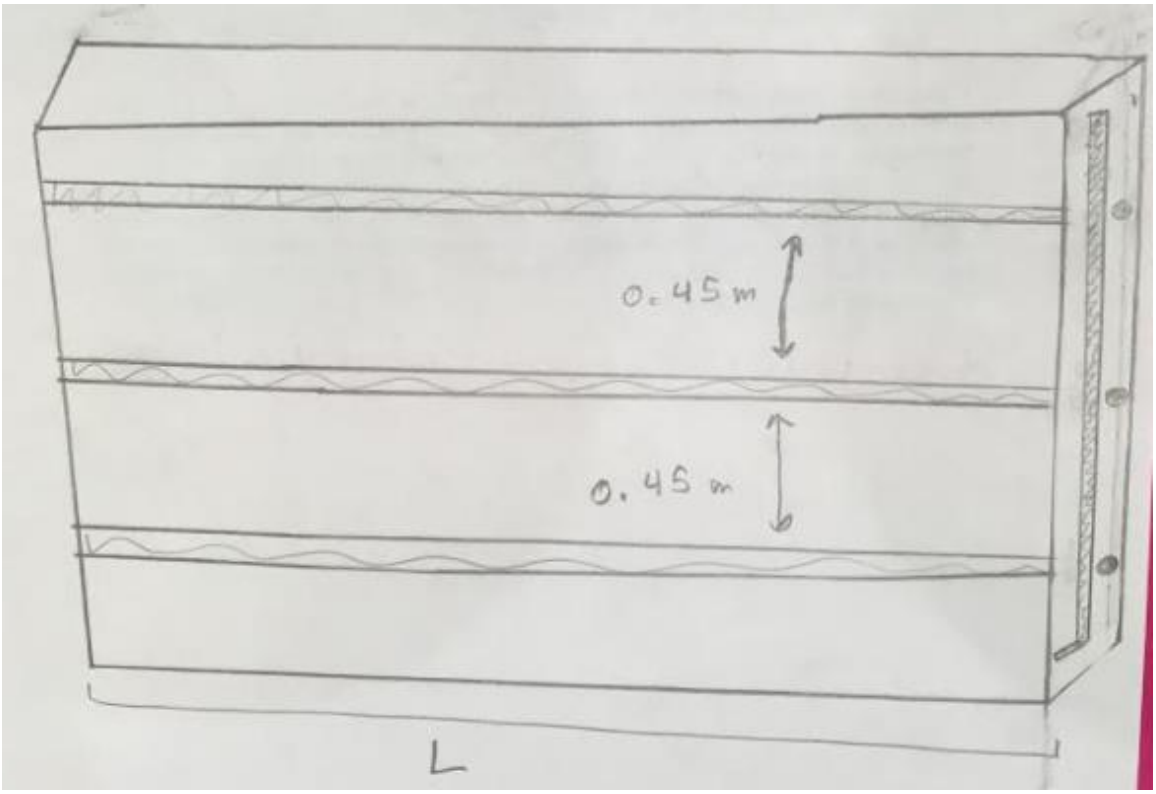

Figure C.5 A sketch of the placement of the reinforcement bars looking at a wall of length "L"

Table C.4 Table of final cistern dimension and reinforcement values

Final Values

Q (m^3/sec)	6.35 x 10^{-3}
V_s (m/s)	1.39 x 10^{-3}
A_{plan} (m^2)	5.48
Settling Tank Length (m)	4.68
Cistern Width (m)	1.17
Cistern Height (m)	1.30
h (m)	1.00
H_w (m)	0.98
d (in)	6.37
Wall Thickness (m)	0.25
Inlet Length (m)	1.6
Outlet Length (m)	1.6
A_s (in^2/ft)	0.0168
Pieces of Vertical Rebar Needed	44
Pieces of Horizontal Rebar Needed	12

Appendix D - Elevation Profiles

Please see figure 17 and tables 2 and 3 for location details.

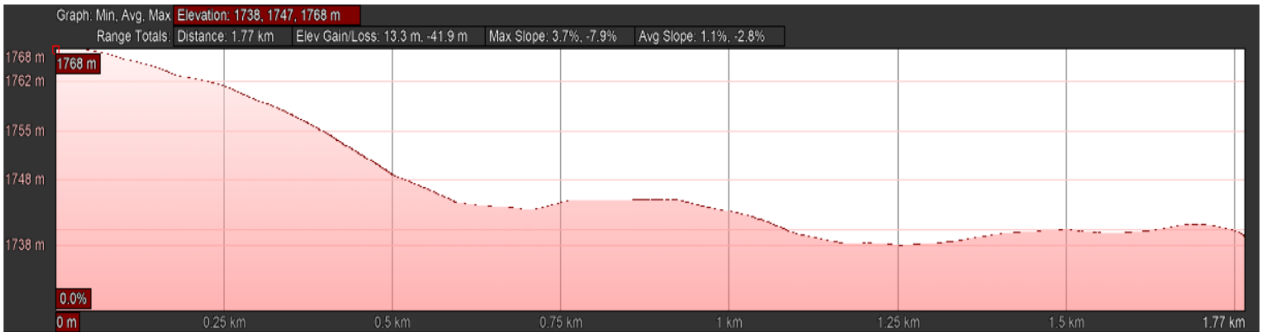

Figure D.1: Source (1) to phase 2 tee (2)

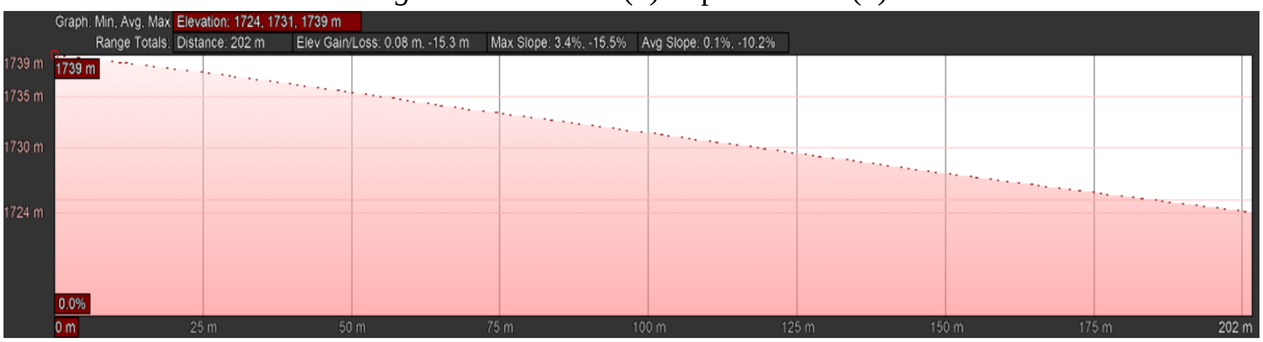

Figure D.2:Phase 2 tee (2) to dist. pts. tee (3)

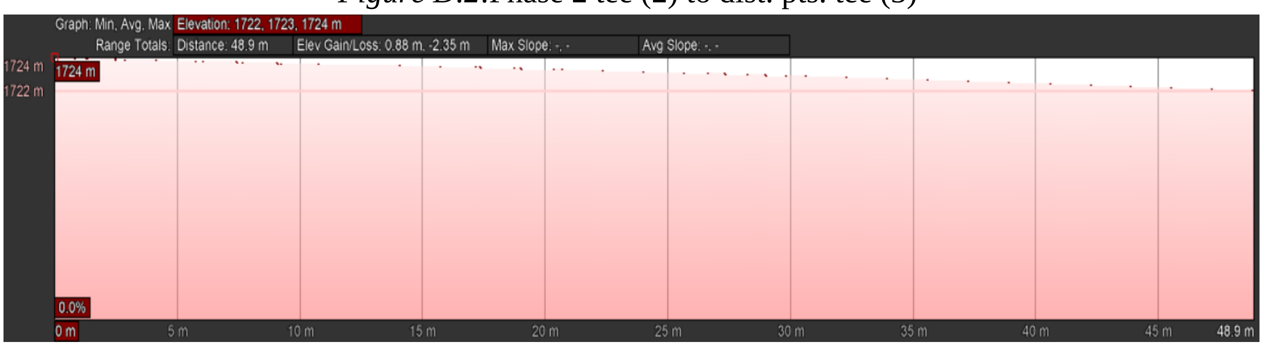

Figure D.3: Dist. pts. Tee (3) to dist. Pt. 4 (4)

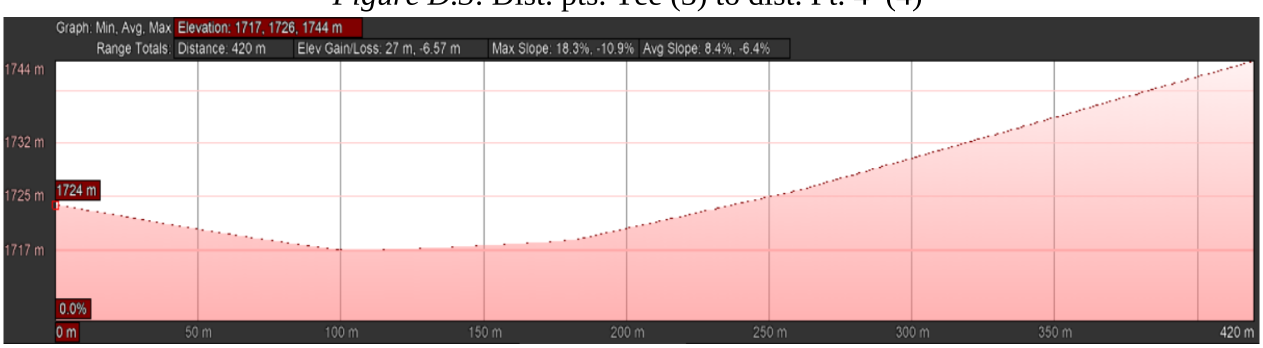

Figure D.4: Dist. pts. Tee (3) to dist. Pt. 5 (5)