Water Tower Frame Design Using Locally-Sourced Wood in Rural Ecuadorian Villages

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ABSTRACT: *Rural community water supplies often require the construction of a village water tower. Safe tower design requires careful planning and properly selected materials. In remote villages, transport of concrete or steel materials is cost prohibitive, and locally sourced construction materials are required. This paper presents a case study of a remote Ecuadorian village in the Amazon, which used locally available hardwood (referred to locally as "Ecuadorian ironwood") for water tower construction in conjunction with a local aid-organisation. Typical samples of the wood were obtained and important structural properties measured. Wood identification was attempted, but matching mechanical properties and optical microscopy imaging to known woods was inconclusive. Using international building codes, an existing tower design was evaluated for principal failure modes. As the tower was originally designed without verifying the material's mechanical properties, the tower was found to be over-designed. By considering water flow requirements for various village sizes, this paper shows how the overall tower height, material thickness, and overall material quantity can be reduced while still maintaining adequate factors of safety. By reducing the amount of required material, villager effort and time are significantly reduced in harvesting the required wood from the forest. The measurement and design process in this paper could be replicated for other rural development projects which rely on locally available, but possibly imperfectly characterised, materials for engineering projects.*

KEYWORDS: Rural water towers, Ecuadorian wood, rural water systems, material characterisation, structural analysis

1 INTRODUCTION

Sufficient access to clean water remains an important challenge for the developing world. Clean drinking water was reflected within two goals of the United Nations (UN) Millennium Development Goals (United Nations, 2015). While assessment data from the UN in 2012 indicate improvements in access to improved drinking water worldwide, the gains are not equitably distributed, and many still lack access to improved water sources (Kiyu and Hardin, 1992). As such, clean water access has been reiterated as Goal #6 of the UN's post-2015 Sustainable Development Goals (SDGs) (United Nations, 2015).

This paper presents details of an engineering effort focused on water systems in rural villages in the Ecuadorian Amazon. Students and faculty joined with Reach Beyond (a non-governmental organisation (NGO) working in Ecuador) to assist rural villages with implementing water systems. Specifically, this effort focuses on the engineering of a water storage tower, a common item of infrastructure required for community water projects.

The specific means of delivering clean water from source to community can take different forms. Gravity-fed water systems are common in rural communities due to their low cost, minimal maintenance, easily controlled pressure, and general robustness and reliability (Arnalich, 2010). One survey of rural community water systems shows that, of almost 1,000 families surveyed, gravity-fed water systems are far more reliable and result in significantly higher user satisfaction than rainwater catchment or mechanical pump strategies (Kiyu and Hardin, 1992). However, a purely gravity-driven system requires the right topology, and in many cases, water must be pumped, at least intermittently, into an elevated tank to allow gravity-driven water flow to the community (Arnalich, 2010).

Many literature resources describe the engineering associated with gravity-fed water projects. In the 1960s, Fair et al. developed a detailed two-volume treatise on water and wastewater engineering, however it lacks an explicit focus on development (Fair et al., 1966). In 1980, Jordan developed a practical handbook on the engineering of gravity-fed water systems for the developing world in conjunction with the UN and the government of Nepal (Jordan, 1980). More recently, detailed technical handbooks exist, which focus on engineering water supplies for small and rural communities (Arnalich, 2010; Smet, 2002; Mihelcic et al., 2009). The United States Environmental Protection Agency (US EPA) has developed an open-source software tool to facilitate design of water distribution systems that can be used effectively in the context of development (U.S. EPA, 2016). Details of specific community water projects are also available (Niskanen, 2003).

While these and other similar sources provide technical information about fluid flow for water projects, there is little to no detail on the actual design of water tower structures. When elevated storage is addressed, there often is an assumption that conventional engineering materials, such as steel and concrete, would be readily available. In the rural Ecuadorian Amazon, transport of such materials is cost prohibitive. Reliance on these types of materials does not fit a definition of appropriate technology, as these materials are not culturally, economically, and socially suitable for the local community (Mihelcic et al., 2009).

The remote nature of these locations means that locally available materials from the forest must be utilised, which presents an opportunity for enhanced sustainability. Local material utilisation means that residents have a greater chance accomplishing maintenance and repair themselves, rather than relying on external support. However, engineering structures, such as water towers, using these materials introduces important challenges.

In the present case, the NGO Reach Beyond presents villagers with a list of materials, based on a tower design, and waits for local community members to harvest and shape the required timber over a period of months. Community members search the local forest for a specific local tree with strength and density, sometimes locally (and unofficially) referred to as "Ecuadorian ironwood". After several months, members of the Reach Beyond team return and assist villagers in constructing the water tower from the timber harvested. Without detailed knowledge of the mechanical properties of the local wood material, ensuring safety requires the tower to be over-designed. The lack of information about tower design in literature, particularly construction with locally sourced wood, increases uncertainty.

2 PROBLEM STATEMENT

To ensure safety with unknown material properties, towers incorporate structural members that are far larger than required to achieve a safe structure. In some cases, using over-sized members can detract from safety by increasing the self-weight of the structure. Over-sized structural members also significantly increase the difficulty in finding suitable trees, as larger trees are less common.

The main objective of this paper is to report a process to determine the required water tower height, material characterisation, and tower re-design as a model that can be applied to similar development projects using poorly characterized local wood material. An analysis is presented to determine required tower height given reasonable community uncertainties and a material analysis of the local, unique wood species is presented. As far as the authors can determine, this local wood species has, to this point, not been characterised in literature. In conclusion, this paper presents a structural analysis that suggests a suitable structural member size that reduces the construction burden.

3 METHODOLOGY

Water flow, material and structural analysis was required for the design of the water tower. The water flow analysis determined the required tower height given data on community population, the material analysis calculated the strength properties of the wood to aid in the material's characterisation, and the structural analysis, incorporating conclusions from both the water flow and material analyses, determined the viable reductions in the size of tower members to facilitate material gathering and construction, whilst maintaining safety.

3.1 Water Flow Analysis

The height of the water tower dictates the size of structural members as well as the water flow that can be obtained. The tower height should be the minimum required to obtain a sufficient flow rate of water. If the tower is made too tall, larger structural members will be required, increasing the volume of wood required to be harvested and increasing construction difficulty. Excessive tower height can result in excessive water flow velocities; flow velocities above 3.0 m/s can cause erosion of pipe from suspended particles in the water over time (Jordan, 1980).

The required tower height will vary by community, and is based on the exact details of the water flow path and community layout. However, analyses using ranges of typical values can give a sense of typical required heights and more importantly, demonstrate design trade-offs and set community expectations.

According to Reach Beyond, a typical water distribution system from tower to individual household involves a 300 m straight flow through 50 mm (nominal) polyvinyl chloride (PVC) pipe, followed by another 300 m run through 40 mm (nominal) PVC pipe. From this main supply line, the branch flow to each household is through a 20 to 100 m run of 32 mm (nominal) PVC pipe. The actual internal diameters of 50, 40, and 32 mm (nominal) PVC pipe are 42.6, 34.0, and 27.2 mm, respectively.

A tower must elevate water high enough to overcome friction in the distribution system, and to provide adequate flow rate at the household spigot. In typical U.S. households, the EPA rates high-efficiency water taps at a maximum flow rate of 3.8 L/min (1.5 U.S. gallons per minute (GPM) and conventional (kitchen sink) water taps at a maximum 3.8 L/min (2.2 GPM) (U.S. EPA, 2014). Jordan uses a value for typical flow rates for rural water systems as 0.225 litres per second (LPS), equal to approximately 3.6 GPM (Jordan, 1980). Given these values, in this study, a base value of 0.139 LPS (2.2 GPM) was used, equivalent to a typical water tap (or faucet) in the United States at maximum flow, and the analysis was carried out for 50 to 150% of this estimated target flow rate.

Once a target flow rate is chosen, the frictional losses can be determined from the layout of the water distribution system. Friction factors for internal pipe flow are well known and developed in textbooks on fluid dynamics (Munson et al., 2006). Along a straight pipe, the water flow for this type of application is usually turbulent. In that case, the friction factor is a function of Reynolds number and non-dimensional pipe roughness. The friction factor can be calculated by implicit solution of the Colebrook equation:

$$\frac{1}{\sqrt{f}} = 1.14 - 2.0 \log\left(\frac{\varepsilon}{D} + \frac{9.35}{Re\sqrt{f}} \right) \qquad [1]$$

where:

f is the friction factor (-)

Re is the Reynolds number (-)

ε/D is the non-dimensional pipe roughness (-)

ε is the absolute roughness (mm)

D is the internal pipe diameter (mm)

A typical absolute roughness for PVC pipe is 0.0015 mm. For very low required flow rates (e.g. small communities), the internal pipe flow can become laminar, in which case the friction factor is a function only of the Reynolds number. The flow friction is dominated by these major losses, but minor losses for flow through valves, diameter changes, bends, and t-sections were also considered.

Typical and representative piping connections were assumed: 3 short-radius 90° elbows (loss coefficient K_L = 1.5) in the initial two pipes (50 and 40 mm nominal), one t-section in line flow (K_L = 0.90), and one 90° ball valve in the household branch flow. A step reduction (K_L = 0.05) was included between pipe diameter changes. While the specific details of the flow may change among villages, these values provide an approximate baseline for design.

3.2 Material Strength Analysis and Identification

The material analysis has two goals, to determine the specific material properties needed in the structural analysis and to document the properties of this wood. The material properties calculated include the density at given moisture content, the modulus of elasticity, and the compressive, tensile and bending ultimate strengths.

The density of the wood is required to calculate the self-weight of the water tower. As wood is a porous material, its properties are affected by absorbed water. Therefore, the properties are reported relative to the moisture content.

The moisture content was measured in accordance with American Society for Testing and Materials (ASTM) standard D143-14 in which samples were weighed before and after being oven-dried at a temperature of 103 ± 2°C (ASTM, 2014). The moisture content x (%) was then calculated using Equation 2.

$$x = 100\left[\frac{m_M - m_0}{m_0}\right] \qquad [2]$$

where:

x is the moisture content (%)

m_M is the initial mass (g)

m_O is the oven-dry mass (g)

Note that this definition for moisture content will allow values over 100%, which would indicate that the wood matrix is holding a greater mass of water than the wood itself. The Wood Handbook by the U.S. Department of Agriculture indicates values of 200% are common with green wood (Forest Product Laboratory, 2010).

The density of the wood (ρ_x) at a given moisture content x was determined by first calculating the specific gravity (G_0) of the oven-dried mass. The specific gravity was measured using the volume by water immersion method ASTM D2395-07 and calculated using Equation 3 (ASTM, 2007):

$$G_O = K \frac{m_O}{V} \quad [3]$$

where:

G_0 is specific gravity (-)

m_O is the oven-dry mass (g)

K is the factor related to standard density of water = 1.00 cm³/g

V is the volume of the specimen (cm³)

Density calculations were also conducted by alternate means using linear measurement of constructed specimen cubes, with reasonable agreement.

Finally the density of the wood (ρ_x) at a given moisture content (x%) was calculated using Equation 4.

$$\rho_x = \rho_w G_O \left(1 + \frac{x}{100}\right) \quad [4]$$

where:

ρ_x is the density of wood at a given moisture content (g/cm³ or kg/m³)

ρ_w is the standard density if water (1 g/cm³ or 1000 kg/m³)

G_0 is specific gravity (-)

x moisture content (%)

Equations 3 and 4 follow the development in ASTM Standard D2395-07a (ASTM, 2007) and the Wood Handbook (Forest Product Laboratory, 2010) and allow density to be calculated for wood specimens of variable moisture content.

The maximum compressive axial strength (σ_c) measures the material’s capacity to withstand compressive loads. The longitudinal modulus of elasticity, E_L, measures the stiffness of the material. Both properties were determined from a compression test in accordance with standard ASTM D4761-13 (ASTM, 2013). A total of 12 specimens were tested to failure at a strain rate of 0.032 (mm/mm)/min. Each specimen was approximately 25.4 mm x 25.4 mm x 101.6 mm.

The yield tensile strength (σ_y) and the ultimate tensile strength (σ_{ultT}) measure the material’s capacity to withstand tensile (pulling) forces before undergoing permanent damage and ultimately rupturing. Both properties were determined using a tensile test where a load applied at the ends pulls the specimen to failure. A total of 20 specimens were tested at a strain rate of 0.026 (mm/mm)/min in accordance with standard ASTM D4761-13 as shown in Figure 1 (ASTM, 2013). The geometry of each specimen was in accordance with standard ASTM D143-14 except for the clear distance between the grips (ASTM, 2014). The amount of wood obtained was limited in quantity and size; as a result, the clear distance could not adhere to the minimum of 25 times the width specified and a clear distance of 16 times the width was instead used.

The ultimate bending strength, σ_{ultB}, measures the material’s capacity to bend before rupturing. It was

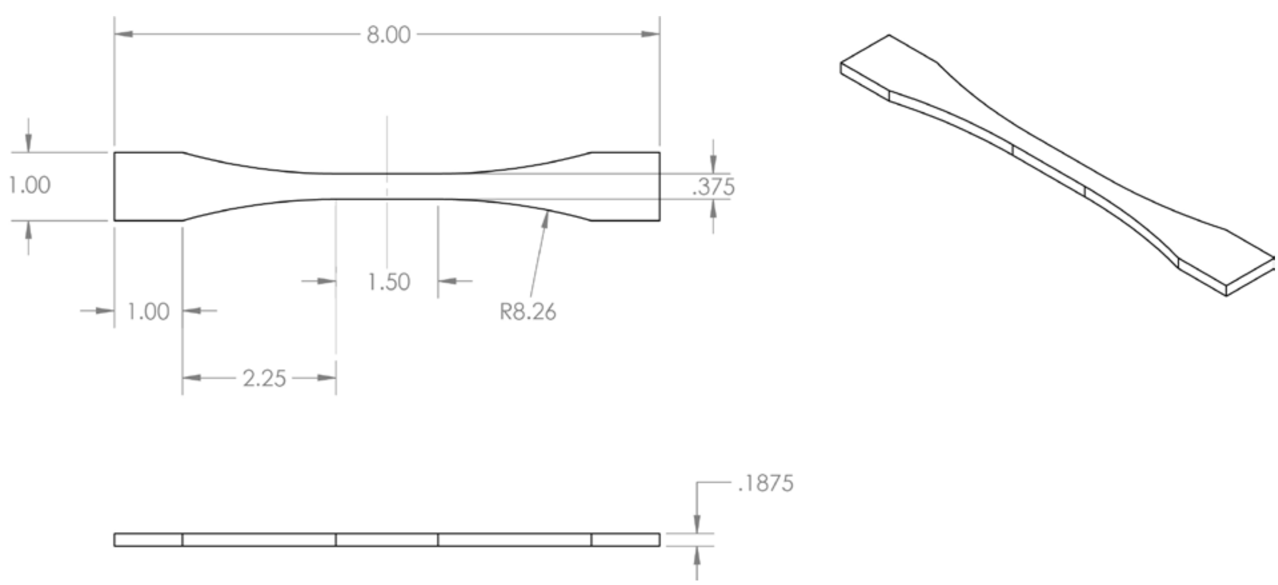


Figure 1: Tensile test specimen design (dimension in inches)

determined from a four-point bending test in accordance with standard ASTM D4761-13 (ASTM, 2013). The test apparatus shown in Figure 2 applied a load until failure at a strain rate of 0.1 (mm/mm)/min. A total of 10 specimens were tested, each measuring 11.2 mm x 11.2 mm x 190.5 mm.

In addition to characterising the wood through material strengths tests, the wood was also categorised as either a hardwood or softwood using microscopy. Under optical microscopy, hardwoods show pores and vessels for water conduction.

Further classification for hardwoods observes the pore size transition from early-wood to late-wood within the growth ring, these transitions can be either ring-porous, semi-ring porous or diffuse-porous. Samples of the wood were cut into cubes of 25.4 mm x 25.4 mm x 38.1 mm and sanded at 120, 150, 220, 320, and 600 grits. The specimens were viewed at 500x and 2000x magnification with an optical microscope. Specimens were also sent to the U.S. Department of Agriculture (USDA), Forest Products Laboratory (Madison, WI) to attempt identification.

3.3 Structural Analysis

The objectives of the structural analysis were to analyse the structural performance of the current water tower design, and to reduce the member sizes, thus reducing construction difficulty and self-weight.

The original tower design is approximately 5 m wide and 7 m tall with primary columns of 20 cm x 25 cm cross-sections and beams and braces with 15 cm x 20 cm cross-sections. The general tower design is shown in Figure 3. The four tower legs are embedded in concrete and all connections are rigid, preventing rotation at the joints.

The loads on the tower include the self-weight of the structure, the water load of 98.1 kN from a maximum load of 10,000 L of water, and a wind load of 525 Pa. The wind load was determined to code set by the American Society of Civil Engineers (ASCE) and takes into account the dimensions of the tower, the wind speed in the region, the use of the structure, and the landscape surrounding the structure (ASCE, 2003). Due to the building type classification and risk category, adverse winds like hurricanes are not considered. The magnitude of the wind load is nearly an order of magnitude smaller than the water and self-weight loads; thus the wind is expected to have a negligible effect on the structural performance of the tower.

In accordance with ASCE 7 Section 9 and the International Building Code (IBC) Section 23, earthquake loading was not considered because the tower is not more than two stories tall and is a storage structure intended only for incidental human occupancy (International Code Council, 2012).

Figure 2 (top): Testing setup for four-point bending test

Figure 3 (bottom): Original water tower design, under construction in a village, showing general size and number of structural members

Due to the risk category and classification of the building, point loads were not investigated with accordance to ASCE7 and IBC.

These conditions were then translated into an analytical and numerical model. In both models the water was assumed to be equally distributed along all four sides, the wind load was modelled as a uniform pressure, and the connections were rigid. Because of the symmetry of the tower, the analytical model was simplified to a 2D-frame analysis, but the numerical model represents the full 3D structure of the tower.

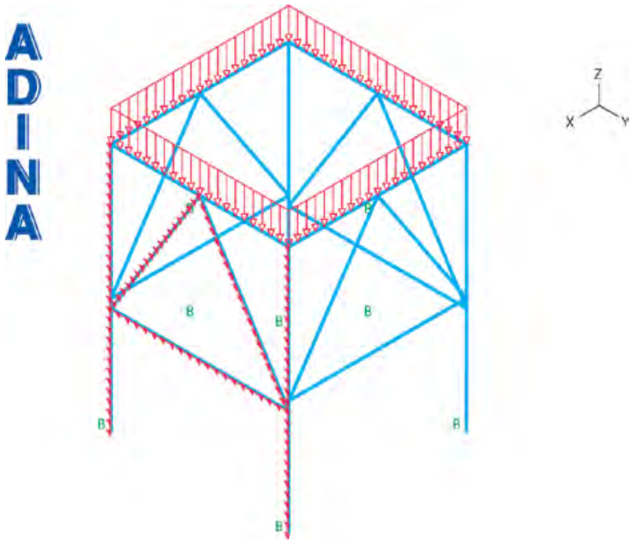


Figure 4: Finite element model used for full analysis of the water tower structure using the software package ADINA

The finite element model shown in Figure 4 was built and analysed using ADINA. ADINA is a commercial software package for finite element analysis of solids and structures (ADINA, 2015). The members were discretised with 2-node Hermitian beam elements. The models were then analysed to assess the strength, stiffness, and stability of the current structure.

Strength was checked by calculating the member stresses due to the applied loads and comparing them to the material stress limits. In the finite element model, the stresses were calculated and solved using ADINA.

In the 2D frame analysis, the water was modeled as uniformly distributed on all the top beams. For the columns and braces, the mass was lumped at the connections, and for the beams, the mass was uniformly distributed along their lengths.

Analytically, the stresses were calculated using fundamental mechanics. The normal stress is the result of stress created by the internal axial force and the bending moment as shown in Equation 5 below.

$$\sigma = \frac{N}{A} + \frac{M_y}{I} \qquad [5]$$

where: σ is the normal stress (Pa)

N is the axial force (N)

A is the cross-sectional area (m²)

M_y is the bending moment (Nm)

y is half the height of the cross-section (m)

I is the second moment of area (m⁴)

The maximum normal stress will occur in the top beams and the bottom columns. The normal stress in the top beams will be dominated by the bending moment resulting from the water load and the self-weight as shown in Equation 6 below.

$$M_{beam} = \frac{1}{48} w_w L^2 + \frac{1}{12} w_{sw} L^2 \qquad [6]$$

where: M_{beam} is the bending moment (Nm)

w_w is the water load (N/m)

w_{sw} is the self-weight (N/m)

L is length of the beam (m)

The normal stress in the bottom columns will be dominated by the axial force N_{col} resulting from the water load F_w and the self-weight of the tower F_{sw}

$$N_{col} = \frac{1}{4}(F_w + F_{sw}) \qquad [7]$$

where:

N_{col} is the normal force (N)

F_w is the axial force resulting from the water load (N)

F_{sw} is the axial force resulting from the self-weight of the tower (N)

Stiffness is evaluated by calculating the deflection of the structure and comparing it to deflection limits set forth by the IBC. Based on the structure's building classification, the maximum deflection allowed is as set out in Equation 8 below (International Code Council, 2012).

$$\delta \leq \frac{h}{180} \qquad [8]$$

where:

δ is the maximum deflection (m)

h is the height of the tower (m)

Lastly, stability ensures the columns will not buckle and is calculated by the Euler Buckling load Pcr for a fixed-fixed column as shown in Equation 9 (Timoshenko, 1961):

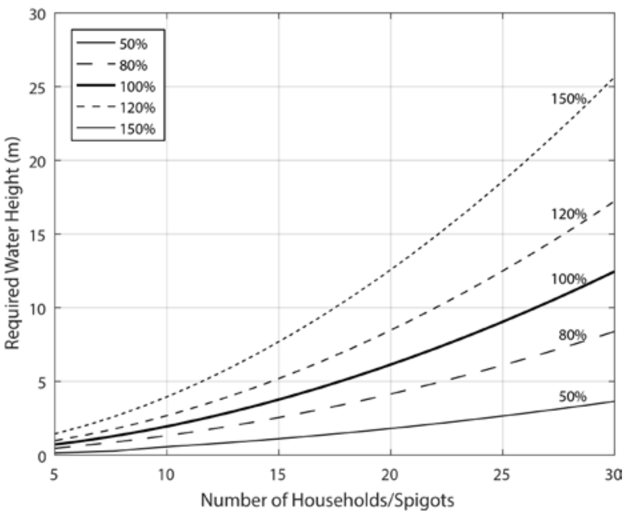
$$P_{cr} = \frac{\pi^2 E I}{(0.5\,L)^2} \qquad [9]$$

where:

P_{cr} is the Euler Buckling load (N)

E is modulus of elasticity (Pa)

L is length of the column (m)



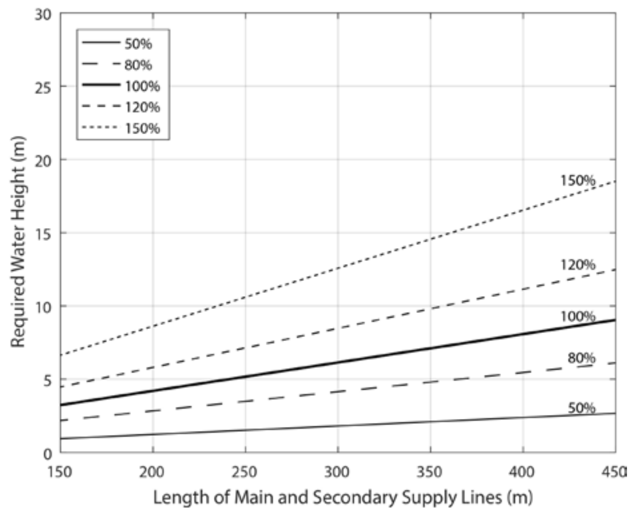


Figure 5 (left): Require water tower height for different community sizes and water flow rates
Figure 6 (right): Required water tower height for different main and secondary pipe lengths and water flow rates

4 RESULTS

4.1 Water Tower Height

Figure 5 shows the results of the fluid flow calculations. The figure gives required water tower height (vertical axis) for different community sizes, indicated by the number of households (horizontal axis), and parameterised by actual flow rate (50% to 150% of the base 8.3 L/min (2.2 GPM target). As shown in Table 1, the calculation assumes that 20% of the households are using a spigot at the same time.

Given that the partner NGO, Reach Beyond, typically works in communities with 20 households, the data suggest that a 7 m high tower is a reasonable height for most scenarios. The effects of various scenarios, such as community expansion, can be simulated using data shown in Figure 5. For example, a 50% community expansion from 20 to 30 households would result in a maximum flow rate decrease to just under 80% of the design condition with a fixed tower height of 7 m. Likewise, an increase in the number of households using the system at the same time (above the 20% estimate used in this model) can be simulated by simply sliding to the right along the horizontal axis.

Table 1: Water tower height modelling parameters

Design Parameter	Base (Design) Value	Variation
Main line, 50 mm (nom)	300 m	150 to 450 m
Secondary line, 40 mm (nom)	300 m	150 to 450 m
Household branch, 32 mm (nom)	50 m	None
Required flow rate per spigot	2.2 GPM (0.139 LPS)	50% to 150%
Number of households	20	5 to 30
Number of spigots on together	20% of households	None

Figure 6 gives results of a similar calculation that shows the effects of variable main and secondary line pipe lengths with fixed households (20). This calculation shows that the required tower height is less sensitive to variations in main distribution line length from the base design value of 300 m. Again, a 7 m tall tower could accommodate shorter or longer runs of the 50 mm and 40 mm (nominal) pipe with modest changes in the available flow rate.

4.2 Material Mechanical Properties and Identification

The experimental tests for mechanical properties yielded the data shown in Table 2 for ultimate strength and modulus of elasticity in compression, tension, and bending. These tests were conducted for specimens with a (typical) moisture content of 7.3%. The wood can be summarised as being hard, strong, and very dense, with a density of 968 kg/m^3.

All three tests (compression, bending and tension) yielded different elastic moduli that is typical of wood. However, the modulus is usually determined from the compression tests (Forest Products Laboratory, 2010). Thus, the following structural analysis calculations use an elastic modulus (E) of 10.3 GPa and use the ultimate bending strength (σ_{ultB}) of 176 MPa and the ultimate compressive strength (σ_c), of 90 MPa for the structural design. For

Table 2: Material strength properties of Ecuadorian wood

Material Property	Ultimate Strength (MPa)		Modulus of Elasticity (GPa)	
	Value	**95% C.I.**	**Value**	**95% C.I.**
Compression	90	88-92	10.3	10.0-10.5
Tension	167	154-180	2.3	2.2-2.4
Bending	176	166-185	35.2	33.8-36.7

reference, the corresponding properties for steel are an elastic modulus of 210 GPa and an ultimate tensile strength of 400 MPa, and for concrete an elastic modulus of 40 GPa, and an ultimate compressive strength of 40 MPa. In comparison to better-known woods like birch and oak, the Ecuadorian wood is stronger, but not as stiff, as shown in Figure 7.

Images from optical microscopy at 500x and 2000x are shown in Figure 8. The presence of pores confirms this wood is a hardwood, and the pore arrangement could be classified as diffuse-porous growth ring pore arrangement with solitary pores.

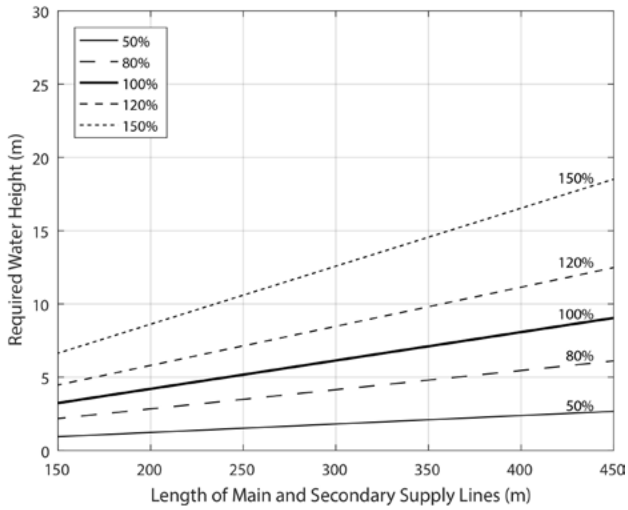
Analysis by the USDA Forest Products Laboratory was inconclusive, but suggested that the wood was likely in the Buchenavia or Terminalia genus. A survey of published mechanical properties from available species in these genera did not reveal a good match to the measured mechanical properties.

4.3 Proposed Structural Design Changes

The results of the numerical 3D frame analysis using ADINA, including the distribution of moments (N_m) and axial forces (N), are shown in Figure 9. The results indicate that the structural members of the original design (20 cm x 25 cm columns and 20 cm x 15 cm beams and braces) are larger than necessary. The maximum normal stress in the beams was 5.56 MPa and in the columns it was 2.82 MPa, which are only 3% of the wood’s ultimate bending and compressive strengths (Table 2).

The numerical results from ADINA were compared with the analytical 2D frame analysis presented in Section 3.3. In Table 3, the analytical calculations are compared to the numerical. The values listed are only for the members with the maximum internal axial force and moment that are the bottom columns and the top beams, respectively. The 2D frame analysis and the numerical results show excellent agreement, and the small differences are due to the simplification of the 3D structure into a 2D frame.

A new design of 15 cm x 15 cm for all members (columns, beams, braces) was proposed and analysed using ADINA in 3D. The resulting stresses were again compared to



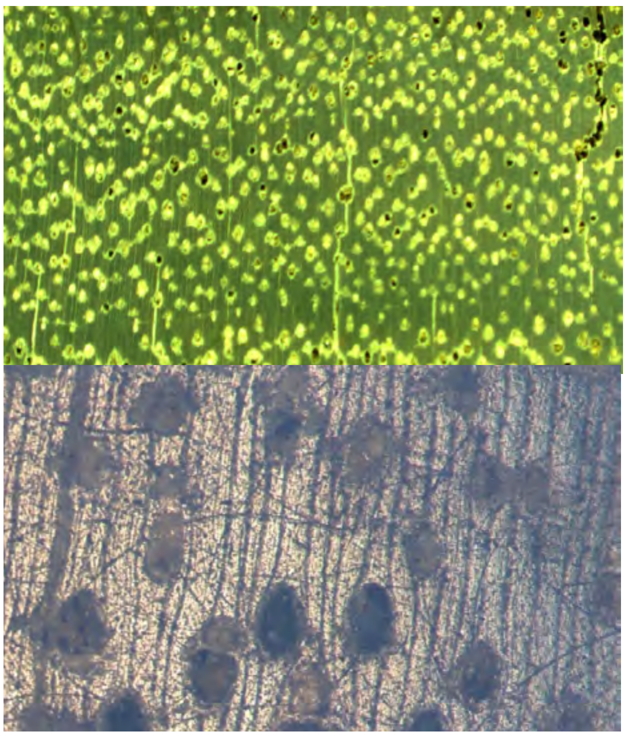


Figure 7 (top): Comparison of the mechanical properties of Ecuadorian “ironwood” with birch and oak

Figure 8 (bottom): Optical microscope images at 500x (left) and 2000x (right) magnification showing pore structure of the Ecuadorian hardwood

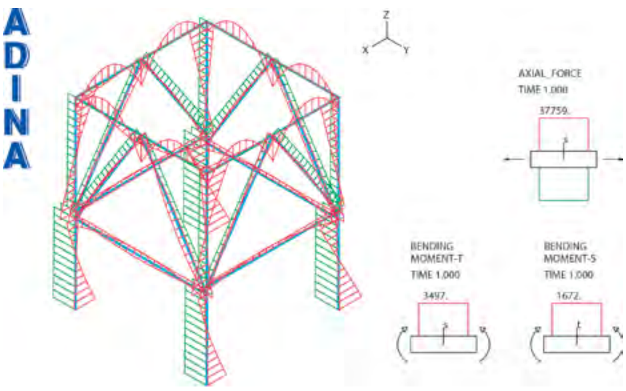


Figure 9: 3D frame analysis results using ADINA showing distribution of moments and axial forces

the wood's strength. The maximum normal stress in the beams was 6.31 MPa and in the columns was 3.36 MPa, again below the material limits (3.7% of the material's capacity). Reducing the size of the cross-sections decreased the weight of the tower by 36%, making it easier to construct and to transport the logs from the forest to the village. Such a reduction also increases the number of trees suitable for the tower.

The final factor of safety of 14.2 is defined by the beam normal stress (6.31 MPa) against the measured ultimate compressive stress (90 MPa). The calculated deflection of the redesigned tower was 0.45 mm, well below the building code limits of 38.9 mm.

The load to buckle the columns was 4.24 x 106 N (Equation 9) that is well above the maximum compressive axial force (3.06 x 104 N) seen by the tower, therefore the new design will not buckle.

5 CONCLUSIONS

This paper presents an engineering process to help improve the design of water towers built with locally sourced, but poorly characterised materials in rural Ecuador. Using materials such as concrete or steel is prohibitive in such communities; in addition, using local materials can help empower the local population to continue to maintain the water tower structure.

The dearth of information on the specific Ecuadorian tree used in these projects leaves the NGO with little option but to over-design, resulting in a tower that is much more robust and sturdy than necessary for safety and function. Excessive size extends the required time for material collection, due to the need to find larger trees and carry greater weight, and also extends the time for construction. Both factors reduce the number of projects the NGO can initiate, limiting impact. Excessive self-weight of the structure could also impact safety during construction.

In this effort, the need to analyse and re-design the water tower was performed with three approaches. First, a water flow analysis used the best available data about local villages, from the NGO group that works in the communities, to determine ranges for the required height of a water tower. The analysis shows that a tower approximately 7 m tall will often be sufficient for typical communities. Second, the unique local wood was characterised by standard methods for density, moisture content, strength, and stiffness. The wood is unique to the area, with a high density and strength, and less stiffness, when compared with traditional North American hardwoods. The acquisition of more samples will facilitate species identification, greater material characterisation, and comparison with other local wood species.

The basic tower design in use was analysed using both simplified analytical and numerical approaches. Results indicated that the current tower design specified beam members that were excessively large in cross-section. These approaches suggested a reduction in member sizes to 15 cm square, resulting in a reasonable factor of safety of 14 and an overall weight savings of 36%. Not only does this reduce the self-weight of the resulting structure, it greatly reduces the time necessary to harvest the wood, and increases the flexibility of NGO teams in finding beams without errors or flaws. These effects result in an overall reduction of construction time required to build the tower and increases the impact of the NGO development organisation.

In practice, Reach Beyond has successfully adapted these recommendations for smaller beam sizes and found improvements in tower constructability. The analysis and methodology presented in this paper also represents a template for addressing other engineering challenges in rural development.

Table 3: Comparison of analytical and numerical axial forces and moments of the original tower design

Parameter	Analytical	Numerical
Axial force – Bottom Column (kN)	33.6	34.1
Moment – Top Beam (kN-m)	3.15	3.24

6 ACKNOWLEDGEMENTS

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