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Managing Trans-Regional Water Transfer in Dry African Countries to Mitigate Shortages *(pp. 249-273)*

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Abstract: The deficiency of water resources is limiting directly and severely the socio-economic development and ecological environmental protection in the Internal Drainage basin (IDB) in the central part of Tanzania which is considered to be semi-arid region. Due to the regional differences in water availability a revised setting of in country water distribution is required. The objective of the work described here within is to determine the least-cost solution for a national water transportation system among the regions, simultaneously meeting the water delivery requirements in the IDB and complying with all other relevant regulatory criteria. The principal of optimal water transportation from eight administrative source basins to the IDB by cost minimization has been implemented. The results obtained by using 70% level of annual renewable water for Environment Water Requirements (EWR) suggest that excess annual renewable water resources from source basins is sufficient to supply the IDB. Transportation constraints imposed in the water management model limits some of the basins to be considered as preferred water sources. On the basis of the modeling results, Lake Tanganyika basin, Lake Rukwa basin and Wami/Ruvu river basins (all are in Tanzania) have been identified as reasonably preferred water sources. Pangani basin in the north-eastern part of Tanzania is under water shortage, water withdrawal from the Pangani basin would further damage the Pangani river ecology and water environment. The optimum total costs (operation costs, capital investments and maintenance of the concrete pipeline) range between 0.69 and 1.37 US $\$/m^3$.

Key words: water allocation, optimum total costs of water transportation, management model, Tanzania, Internal Drainage Basin, environment water requirements

NOTATIONS

Definition of symbols and abbreviations used in models are given in Table 1 while additional symbols are defined as they appear.

Table 1: Nomenclature

Ar	Arusha sub-basin	Ruv	Ruvuma basin
C	Hazen-Williams coefficient (dimensionless)	V_i	Total annual volume of water (m^3/yr) that can be supplied from a given source basin *i*
C_{ij}^{T}	Cost (US \$/m^3) of water transfer from source basin *i* to water delivery location *j*	Sh	Shinyanga sub-basin
C_{i}^{W}	Cost (\$/m^3) of water treatment that depends on water quality index (WQI) of source basin *i*	Si	Singida sub-basin
CM_{ij}	Capital investiments and maintenance costs (US \$/year) from source basin *i* to water delivery location *j*	Tab	Tabora sub-basin
D_j	Total annual water delivery requirement (m^3/yr) at a given location *j* in the IDB	Tan	Lake Tanganyika basin
Do	Dodoma sub-basin	T	Period of pumping (hrs) per year
EAC_j	Equivalent annual cost (US \$/year) at location *j* in the IDB	Vi	Lake Victoria basin
EWR	Environmental water requirement (m^3/year)	Wa:	Wami/Ruvu basin
HP	Energy (horsepower) requirement of pump	*WQI*	Water quality index (dimensionless score ranging between 1 to100)
IDB	Internal Drainage basin	X_{ij}	Volume of water (m^3/yr) diverted from source basin *i* to water delivery location *j*
J	Gradient of energy expressed in promille (part per thousands) (‰)	ΔH_{ij}	Total required head (m) (sum of elevation head Δz_{ij}, and pipe friction losses Δhf_{ij}) to raise water from source basin *i* to water delivery location *j*
L_{ij}	Distance (m) between the source	Δhf_{ij}	Pipe friction losses (m) from source basin *i* to water delivery location *j*
		Δz_{ij}	Topographic elevation (m) at which water has to be raised from the source basin *i* to the water delivery location *j*
		β	Price (US \$/m^3) of treatment for

basin i to water delivery location j	water with very poor quality
Ma Manyara sub-basin	ϕ_{ij} Internal diameter (mm) of pipe from source basin i to water delivery location j
Ny Lake Nyasa basin	λ Price (US \$/kWh) of electricity
P Energy (kWh) requirement of pump	η Pump efficiency (%)
Pa Pangani basin	
Q_{ij} Water flow rate (m^3/h) from source basin i to water delivery location j	
Qv Water quality value (dimensionless value that ranges from 0 to 100)	
Ruf Rufiji basin	
Ruk Lake Rukwa basin	

1 INTRODUCTION

Severe water scarcity in semi-arid regions, has led the traditional methods of water supply to be inadequate. Water transfer from one region to another is becoming acceptable in order to narrow the gap between supply and demand. The South-North Water Transfer Scheme was developed in China in order to ease water problems by transporting water from the Yangtze River in the south to rivers in the north. Water prices in water shortage cities in China range between 0.20 US \$/m^3 and 0.34 US \$/m^3. However, the water prices do not reflect the true cost of water because since the cost of water is subsidized by the government (Zhou and Tol, 2003) as means to slow-down people migration to urban areas. The Egyptian government planed a water pipeline from the Nile River to supply all the cities that lie on the Red Sea Coast. The installation cost per m^3 depreciated on 15 years is 0.63 US \$/m^3. The running cost including maintenance cost is about 0.44 US \$/m^3. The total cost including installation cost is about 1.07 US \$/m^3 (Djebedjian et el., 2005). Most of the few articles that discuss water transportation costs refer back to Kally (1993). Kally's work does contain a few useful estimates with regard to the costs of transferring water from the Nile river to Gaza which is about US \$ 0.06/m^3 per 100 km by canal. Kally (1993) provides the effects of soil type and transfer mode on costs. The Nile - Gaza transfer is by canal in soft but stable soil. If the soil is rocky, transport costs would be 13% higher, and if the soil is sandy, costs would be 175% higher. Transporting water by pipe would lead to a cost increase of 271%, while a tunnel would cost 108% more than a canal (Kally, 1993).

Water transportation is commonly in order to mitigate regional shortage which is due to low precipitation and/or poor groundwater sources.

One of the first attempts to formulate and solve models referring to large scale water supply systems was made by Loucks et al. (1967), using linear programming. Later, a wide variety of approaches were applied: Graves et al., (1972) used nonlinear programming; Brill and Nakamura (1978) used linear mixed-integer programming with branch and bound method; Cao et al. (2007) used a pinch multi-agent genetic algorithm for optimizing water using networks. Problems involved in regional water transportation systems planning can mainly be dealt properly if the corresponding optimization model includes nonlinear cost functions for the installation, operation, and maintenance of pipelines, taking as well into account the nonlinear hydraulic flow behavior of pipe networks. Until the 1980s optimization procedures were based mainly on heuristic methods leading mainly to local optimum solutions (Cunha et el., 2009). Since the 1980s, advanced methods often inspired by natural processes have been applied, yielding global optimum surveys of modern optimization procedures (Michalewicz and Fogel, 2004, and Savic and Walters, 1997).

2 PROBLEM PRESENTATION AND BACKGROUND OF STUDY

The United Republic of Tanzania (here refereed as Tanzania) has nine administrative river basins whereby one of them [the Internal Drainage Basin (IDB)] lies in the semi-arid region. The nine administrative river basins defined are (Figure 1): Pangani (Pa) basin, Wami/Ruvu (Wa) basin, Rufiji (Ruf) basin, Ruvuma (Ruv) basin, Lake Nyasa (Ny) basin, Internal Drainage Basin (IDB), Lake Rukwa (Ruk) basin, Lake Tanganyika (Tan) basin, and Lake Victoria (Vi) basin. Due to the regional differences in water availability a revised setting of in country water distribution is required. It can be mainly be solved by defining a management model that will take into account regional water shortages *vs.* the existence of several sources of excess annual renewable water (annual runoff and groundwater).

A management model was developed allowing determining the least-cost for water transportation between regions in Tanzania, simultaneously meeting the water delivery requirements in the IDB and complying with all other relevant regulatory aspects (e.g. minimum pipes diameters, maximum flow in the pipes etc.). It is indeed a conceptual model (not a design) that allows a preliminary cost estimate of in-country regional water transfer to mitigate scarcity in the IDB. The management model provides guidelines and directions to meet anticipated future scarcities. Trans-boundary water transfer between countries is irrelevant due to the worldwide water shortages and optional political complications.

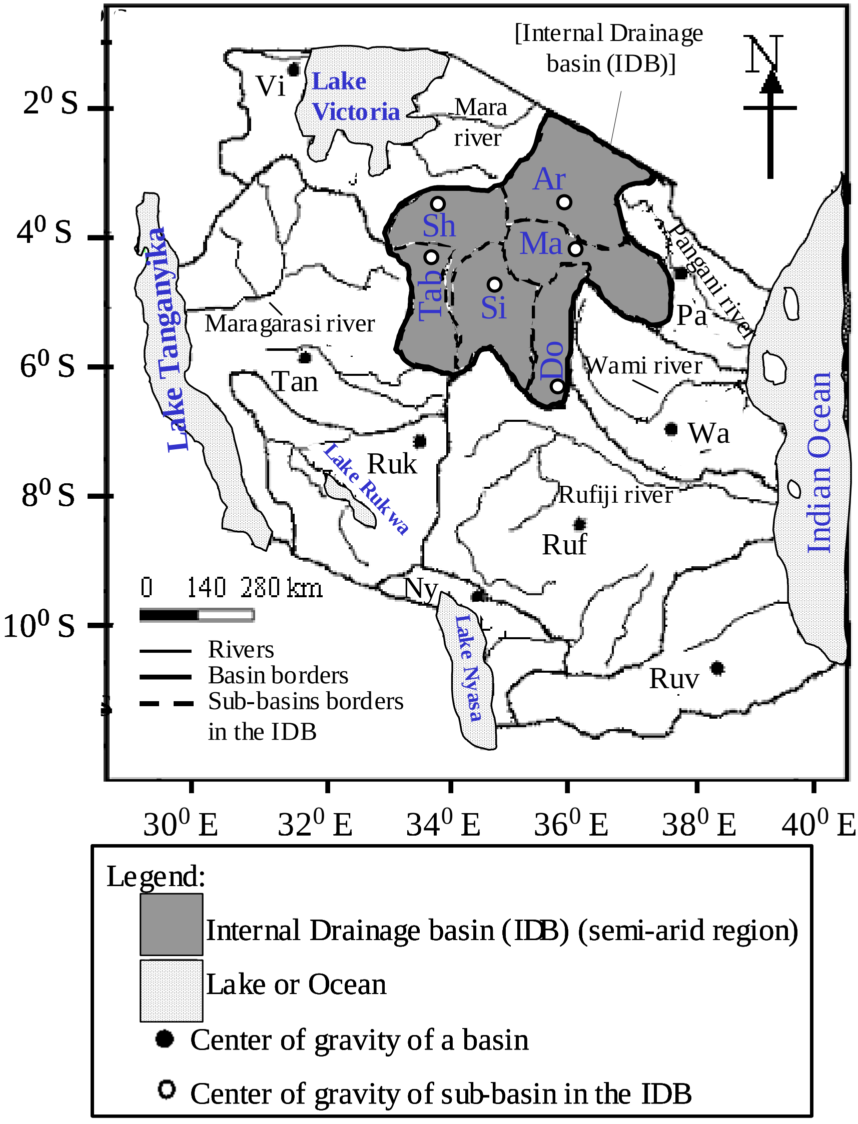


Figure 1: The nine administrative basins in Tanzania: Pangani (Pa), Wami/Ruvu (Wa), Rufiji (Ruf), Ruvuma (Ruv), Lake Nyasa (Ny), Lake Rukwa (Ruk), Lake Tanganyika (Tan), Lake Victoria (Vi), and Internal Drainage (IDB) including sub-basins [Arusha (Ar), Manyara (Ma), Dodoma (Do), Singida (Si), Shinyanga (Sh), and Tabora (Tab)].

Lake Victoria actually supplies water to several countries including the Nile River however, the decline in water availability (water level decrease of 2.5 m from 1999 until 2007) turns it into a non reliable long term water source (Kahumba, 2007; Minakawa et al., 2008).

3 METHODOLOGY

Model Development, Description and Assumptions

The Trans-regional (basin) water transfer in Tanzania involves many factors such as topography and distances, water quality, domestic and industrial water supply, storage, livestock water requirements, irrigation, and water demand for "green" (environmental) purposes. For the purpose of efficient water distribution in the IDB, it was considered reasonable to divide the IDB into six sub-basins [Arusha (Ar), Manyara (Ma), Dodoma (Do), Singida (Si), Shinyanga (Sh) and Tabora (Tab) in which the region stretches] (Figure 1). It is sufficient at this stage to aggregate the time step in the model on annual basis. It is assumed that seasonal changes can be compensated by construction of storage facilities.

"Center of gravity" of source basins and sub-basins in the IDB were defined (Figure 1) in order to estimate the distances and differences in elevations between water delivery locations and the source basins. For the source basins, the area with high probability that a dam for water storage during rain season may be constructed, namely, downstream of main river, was considered as a center of gravity. In case of sub-basins in the IDB, the area with highest water delivery requirements was regarded as the center of gravity, namely an area with large irrigation project or high population. Thus, the objective of the problem is to minimize total cost associated with transportation C_{ij}^{T}, water treatment C_{i}^{W}, capital investments and maintenance expenses $CM_{ij.}$ which are described in the following sections.

Cost of Water Transportation (C^T)

The cost of transporting water is primarily due to pumping (i.e. energy costs). Let us consider total required head (local losses due to bends, valves, etc, are neglected), ΔH (m) to be given in general:

$$\Delta H = \Delta z + \Delta hf \tag{1}$$

where,

Δz = difference in elevation (m) between the source basin to the water delivery location.

Δhf = pipe friction energy losses (m).

Energy requirements for the pump used to convey the water to the water delivery site are given by (Jensen, 1983):

$$HP = Q\Delta H / 2.7\eta \quad (2)$$

where,

HP = the pump power in horsepower (1 HP=0.746 kW) and Q = flow rate (m^3/h).

η = pump efficiency (%). The energy loss Δhf (m), in pipes is due to water flow (friction) and pipe diameter and is proportional to the pipe's length. The gradient of the energy loss in a pipe is usually given by J (part per thousand):

$$J = 1000\Delta hf / L \quad (3)$$

The gradient of the head loss J(‰), due to friction is assessed by Hazen-Williams equation (Jensen, 1983):

$$J = 1.131\text{x}10^{12}(Q/C)^{1.852}\phi^{-4.87} \quad (4)$$

where ϕ is the internal diameter (mm) of the pipe section and C is of the internal pipe smoothness (Hazen-Williams coefficient). The head loss Δhf_{ij} (m), from source basin i to the water delivery site j can therefore be written as:

$$\Delta hf_{ij} = (1.131\text{x}10^{9}(Q_{ij}/C)^{1.852}\phi_{ij}^{-4.87})L_{ij} \quad (5)$$

In this study the pumping time from source basins to IDB is assumed to be around 20 hrs per day. This is equivalent to a pumping period T (hrs) of about 7320 hrs per year. In evaluating the general form of the equation for pressure loss in pipes, the model optimization implements the following assumptions:

Hazen-Williams coefficient for concrete pipe, C = 100 and the pump efficiency η = 65%.

Given these assumptions Equation (5) reduces as:

$$\Delta hf_{ij} = 2.236\text{x}10^{5} Q_{ij}^{1.852}\phi_{ij}^{-4.87}L_{ij} \quad (6)$$

From Equations (1) and (6), Equation (2) can be converted into the required power P_{ij} in kWh as:

$$P_{ij}(\mathrm{kWh}) = \mathrm{Q_{ij}}T(\Delta \mathrm{z_{ij}} + 2.236\mathrm{x}10^5 Q_{ij}^{1.852}\phi_{ij}^{-4.87}L_{ij} \quad)\mathrm{x}0.746 / (2.7\mathrm{x}65)$$
$$= 4.25\mathrm{x}10^{-3}\mathrm{Q_{ij}}T(\Delta \mathrm{z_{ij}} + 2.236\mathrm{x}10^5 Q_{ij}^{1.852}\phi_{ij}^{-4.87}L_{ij}) \qquad (7)$$

The power, P_{ij} (kWh), can be converted into cost per year, Cy_{ij} (\$/yr), given the electricity cost, λ (US \$/kWh) that ranges between 0.05 and 0.15 US \$/kWh, as:

$$Cy_{ij}(\$/\mathrm{yr}) = 4.25\mathrm{x}10^{-3}\mathrm{Q_{ij}}T(\Delta \mathrm{z_{ij}} + 2.236\mathrm{x}10^5 Q_{ij}^{1.852}\phi_{ij}^{-4.87}L_{ij}) \ \lambda \qquad (8)$$

It is know that, $\mathrm{Q_{ij}}T$ is the total annual volume (m^3) transported from source basin *i* to the water delivery location j. Consequently, Equation (8) can be expressed in terms of specific cost (\$/m^3) by dividing by $\mathrm{Q_{ij}}T$. Dividing Equation (8) by $\mathrm{Q_{ij}}T$ (m^3/yr) shows that there is a cost associated with $\Delta \mathrm{z}$ even if the flow $\mathrm{Q_{ij}}$ between a source basin and a water delivery site is zero. In practice, when there is no flow across a link between the source basin and water delivery site the cost for installation, maintenance and operation (energy and treatment) should be zero. To overcome this singularity, when Equation (8) is divided by $\mathrm{Q_{ij}}T$, the elevation term ($\Delta \mathrm{z_{ij}}$) is multiplied by a weight $Q_{ij}/(Q_{ij}+0.00001)$. The term 0.00001 was added to prevent getting singular points with no solution. This negligible term has negligible effect on the solution, however, prevents any mathematical difficulties. Therefore, the total cost per m^3 C_{ij}^T (\$/m^3), is expressed as:

$$C_{ij}^T = 4.25\mathrm{x}10^{-3}(\Delta \mathrm{z_{ij}} Q_{ij} / (Q_{ij} + 0.00001) + 2.236\mathrm{x}10^5 Q_{ij}^{1.852}\phi_{ij}^{-4.87}L_{ij}) \ \lambda \qquad (9)$$

The variable C_{ij}^T is cost of water transportation (\$/m^3) from source basin *i* to the water delivery site *j*, that depends on the distance L_{ij} (m), topographic elevation Δz (m) at which water has to be raised, flow Q_{ij}(m^3/h) and diameter of the pipe ϕ_{ij} (mm). Equation (9) is non-linear with unknown Q_{ij} and ϕ_{ij} which needs to be optimized while Δz_{ij} and L_{ij} are given parameters.

Water Quality and Cost of Treatment (C^W)

One of the components of water delivery requirements in the IDB is for municipal and industrial use. This will need some treatment before supplying to customers. Cost of water treatment depends on the raw water quality at the source basin. For assessment and evaluation of water quality between river basins, the concept of Water Quality Index, *WQI*, was introduced (Irvine et al., 2005). The *WQI* consists of parameters affecting the quality of water by a single score (between 1 to 100). A higher value indicates better water quality. There are many methods of determining *WQI*. However, NSF-*WQI* has gained favor in applications for developing nations where water quality data is scarce. In this work the NSF-*WQI* approach was adopted. In evaluating WQI using NSF approach, the NSF-*WQI* weight (*w*), as well as the water quality value (*Qv*) from rating curves of water quality parameters are used (Irvine et al., 2005). The WQI_i of source basin *i* is computed as weighted average of the *Qv* given by:

$$WQI_i = \left(\sum_{k=1}^{n} w_k Qv_k \right) \Bigg/ \sum_{k=1}^{n} w_k \tag{10}$$

Where,

w_k = weight of water quality component *k*

Qv_k = water quality value for water quality component *k*

n = total number of quality parameters used in the model.

The NSF-*WQI* quality parameters and the corresponding weights (in brackets) used in this study are: Dissolved Oxygen (DO) (0.17), pH (0.12), total nitrates (0.1), and total phosphorus (0.1). The water quality rating curves are available in Irvine et al., (2005).

The indicators for the water quality index (WQI) were assessed with regards to available data in the river basins. These include pH, DO, Total Phosphorus (TP) and Total Nitrogen (TN) (Table 2). In the river basins with scarce data, subjective judgment was made with regards to the information obtained from previous studies (UNEP, 2006; British Geological Survey, 2001). All the basins have the water quality index score of more than 70 (Table 2). This indicates that the basins have good water quality. Details of water quality data used are given in the report by Kazumba (2009).

Table 2: Water Quality Index (WQI) for Estimating Water Treatment Costs by Equation (11)

Basin Name	pH (-)	*Qv*	DO (%) Saturation	*Qv*	TP mg/l	*Qv*	TN mg/l	*Qv*	WQI [Eq. (10)]
Pangani (Pa)	--	--	--	--	--	--	--	--	85*
Wami/Ruvu (Wa)	7.5	93	135	81	1.8	29	0.9	96	76
Rufiji (Ruf)	--	--	--	--	--	--	--	--	87*
Ruvuma (Ruv)	--	--	--	--	--	--	--	--	98*
L. Nyasa (Ny)	--	--	--	--	0.33	78	1.74	95	87
L. Rukwa (Ruk)	--	--	--	--	--	--	--	--	90*
L.Tanganyika(Tan)	9	49	85	98	0.12	95	0.03	97	86
L. Victoria (Vi)	7.1	90	75	81	0	97	0	100	90

-- Missing data, * WQI derived from subjective information

The cost of water treatment Ciw (US $/m^3) from source basin *i*, is defined as:

$$C_i^W = \alpha(1 - WQI_i / 100)\beta \qquad (11)$$

Where α is the fraction of total delivery treatment requirements for use (municipal and industrial) in the IDB. In this case (Table 5) α = 0.26. β is the cost of treatment of very poor quality waters that ranges between 0.05 to 0.15 US $/m^3.

Capital Investments and Maintenance Costs (CM)

In order to define a model that includes capital cost investments, data for grade 100-D of reinforced concrete pipe prices was collected from M CON Pipe and Products Inc (2009). The regression equation between concrete pipe price and pipe diameter shows that for pipe diameter between 300 mm to 3,000 mm the price of concrete pipe *Cp* can be expressed in terms of diameter as:

$$Cp = 0.0009\phi^{1.9043}, \qquad R^2 = 0.99 \qquad (12)$$

Where *Cp* is the price of concrete pipe (US $/m) and ϕ is the internal pipe diameter (mm).

In practice the cost of pipe installation which comprises of excavation, laying and backfilling depends on the soil type. If the soil is soft but stable the cost will be cheaper compared to the rocky soil. However, higher costs can be required when the soil is sandy

(Zhou and Tol, 2005). Experience has shown that the average cost of pipe installation is almost the same as the price of the pipe. Once the concrete pipe is installed, the maintenance expenses are rare and range from 1% to 5 % of the total capital investments. In this study the installation cost is assumed to be the same as the price of the pipe, and the maintenance cost of 5% of total capital investments. Therefore from Equation (12) the annualized capital investments and maintenance cost CM_{ij} (US \$/year) from source basin i to the water delivery location j is given as:

$$\begin{aligned} CM_{ij} &= CRF\,(2\text{x}0.0009\phi_{ij}^{1.9043} + 0.05\text{x } 2\text{x}0.0009\phi_{ij}^{1.9043}) \\ &= 1.89\text{x}10^{-3} CRF\,\phi_{ij}^{1.9043} L_{ij} \end{aligned} \quad (13)$$

Where,

CM_{ij} = annualized capital investments and maintenance costs (US \$/year) in the pipeline from source basin i to the water delivery site j

L_{ij} = distance (m) between source basin i to the water delivery site j

CRF = Capital Recovery Factor given by: CRF= $r/[1-(1+r)^{-n}]$

r = interest rate (fraction)

n = life span of the pipe (years)

It is assumed that the interest rate ranges between 4% and 8% and the life span of the pipes is 40 years. Since the diameter ϕ_{ij} [Equation (13)] is non-zero (diameter in our case ranges between 300 mm to 3000 mm), to avoid cost when the flow is zero during optimization, Equation (13) is multiplied by a weight factor $Q_{ij}/(Q_{ij}+0.00001)$ [Equation (9)]. Hence, Equation (13) in the optimization model is given by:

$$CM_{ij} = 1.89\text{x}10^{-3} CRF\,\phi_{ij}^{1.9043} L_{ij} [Q_{ij} /(Q_{ij} + 0.00001)] \quad (14)$$

Transportation Model Structure

Transportation model involves direct transporting of water from source basin to demand site (Figure 2). Accordingly, the Pan basin is under deficit and the Vi basin has inflow contribution to Lake Victoria of 34% which is also the source of Nile River. These two basins were not considered in the analysis. Provision of 1929 that was signed between Egypt and riparian states (including Tanzania) denied large scale water withdrawal from the Nile including from Lake Victoria except with the prior consent of the Egyptian government. However, there are on-going negotiation for equitably use of the waters through the Nile Basin Initiative (SVP, 2007).

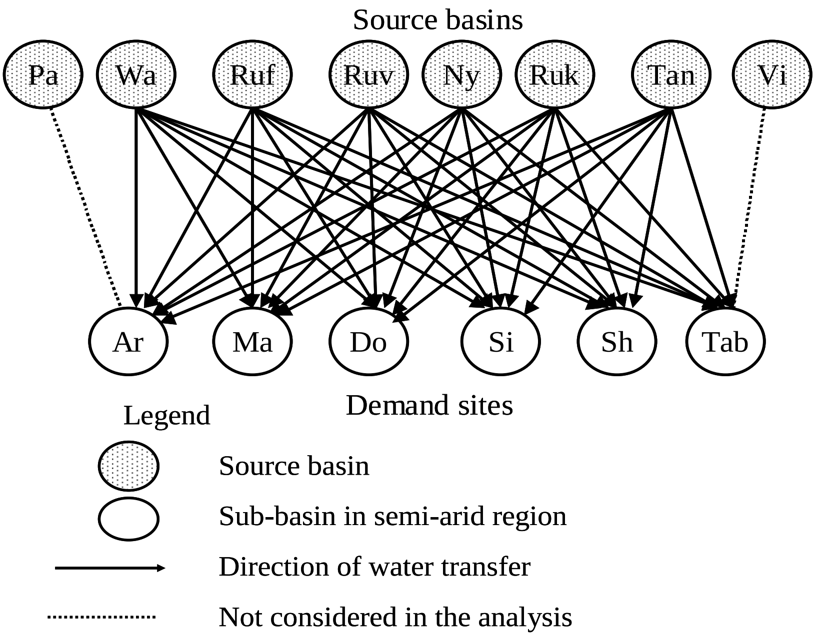


Figure 2: Transportation model layout. [Pangani (Pa) basin is not considered because it is under water deficit and lake Victoria (Vi) basin is a source of Nile river which is the main source of water for other countries downstream consumers]
uirement (m^3/h)] .

Given the situation and objective, the decision variables for water transportation and allocation, a non-linear scenario is represented by Q_{ij}. The variable Q_{ij} is the flow (m^3/h), which is multiplied by the duration of pumping T=7,320 hrs per year to give the volume of water (m^3/year) diverted from source basin i to water delivery location j in the IDB. The economic of the water supply system is described by a cost function (objective function) expressed in terms of an equivalent annual water amount (EAC_j) at each delivery location j. Thus EAC is the total cost of capital investments and maintenance, transportation and treatment:

$$EAC_j = \sum_i \left(C_{ij}^T + C_i^W\right) Q_{ij} T + CM_{ij} \qquad \forall j, \tag{15}$$

where C_{ij}^T, and C_i^W, are variable cost coefficients (\$/m^3) (as defined above) for supplying the water delivery to locations j (j=1,…, n) in the IDB from source basin i

[i=1,…, m] (Figure 2), and CM_{ij} is fixed cost (US $) of capital investments and maintenance. The optimization model takes the form of non-linear (also non-convex) expressions given for n water delivery locations and m source basins:

$$Minimize\ \mathrm{Z} = \sum_{j=1}^{n}\sum_{i=1}^{m}\left(C_{ij}^{T} + C_{i}^{W}\right)\mathrm{Q}_{\mathrm{ij}}T + \mathrm{CM}_{\mathrm{ij}} \qquad (16)$$

Subject to:

(i) Local constraints

$$\sum_{i=1}^{m}\mathrm{Q}_{\mathrm{ij}}T \geq D_{j}, \qquad \forall j, \qquad \text{(water delivery Constraints)} \qquad (17)$$

$$\sum_{j=1}^{n}\mathrm{Q}_{ij}T \leq V_{i}, \qquad \forall i, \qquad \text{(capacity constrints)} \qquad (18)$$

(ii) Non-negativity constraints

$$Q_{ij} \geq 0, \qquad \forall i, j \qquad (19)$$

(iii) Diameter constraints

$$300 \leq \phi_{ij} \leq 3000, \qquad \forall i, j \qquad (20)$$

Here D_j is total annual water delivery requirement (m^3/year) at site j. The parameter V_i is the available water (m^3/year) to export from site i. Minimizing the objective function [Equation (16)] allows to determine the optimal set of TQ_{ij} (i.e. which source basins to use in the country and how much water to transfer to the semi-arid region) and the corresponding optimum pipe diameter ϕ_{ij} (mm). Each continuous pipe diameter can be converted into two consecutive sections of commercial diameters (by taking the head loss of the continuous diameter pipe and equalizing it to same head loss of two successive commercial pipes).

4 COST ASSESSMENT PROCEDURE

The Optimization Algorithm

The expression for the objective function (Equation 16) is non-linear. In the model layout (Figure 2) 36 arcs were considered. Each arc has unknown flow Q_{ij} and diameter ϕ_{ij} to be optimized. The total number of unknown variables is 72, with 84 constraints to be satisfied. The model was solved by using LINGO [version 9.0, LINDO System Inc (2004)] package.

The Global Solver of LINGO was used to obtain global optimal solution. Important procedures used in the Global Solver are as follows:

i. Since the model is non-linear and probably non-convex, other local optima may exist that yield significantly better solutions. The Global solver converts the original non-convex, nonlinear problem into several convex, linear sub-problems.
ii. The Multistart solver in the Global solver was used to generate five set of candidate starting points in the solution space. The nonlinear solver heuristically selects a subset of these starting points to initialize a series of local optimizations for each sub-problem.
iii. For nonlinear models, the primary underlying technique used is based upon a Generalized Reduced Gradient (GRG) algorithm. However, to help getting to a good feasible solution quickly, LINGO also incorporates Successive Linear Programming (SLP). The nonlinear solver takes advantage of sparsity for improved speed and more efficient memory usage. For each sub-problem, local search for nonlinear solvers in LINGO were implemented until they converged to a local optimum.
iv. Subsequently, the Global Solver uses the branch-and-bound technique to exhaustively search over these sub-problems for the global optimal solution until is reached.

Input Data

In order to proceed towards the optimal solution the center of gravities defined for each basin (Figure 1) were used as reference points to measure distances and elevations from source basins to water delivery sites in the IDB. Horizontal distances from center of gravities of source basins to center of gravities of water delivery sites were estimated from maps at the nearest 10th kilometer (Table 3). Due to the limited information of the topographic elevation, adjustment in distances due to topographic variations was neglected. Detailed variations in the routes were difficult to obtain. The differences in elevations were estimated from topographic elevation maps and the estimated differences at the nearest 100th meter are given in Table 3.

Table 3: Distances L (km) and Differences Δz (m) from Center of Gravities of Source Basins to Center of Gravities of Sub-basins in the IDB Approximated to the Nearest 10th kilometer and 100th meter Respectively

From (Water source)	To Water delivery site						
	Distance and Elevation	**Arusha (Ar)**	**Manyara (Ma)**	**Dodoma (Do)**	**Singida (Si)**	**Shinyanga (Sh)**	**Tabora (Tab)**
Pangani basin (Pa)	*L* (km)	270	220	300	330	490	450
	Δz (m)	800	0	0	800	800	800
Wami/Ruvu basin (Wa)	*L* (km)	440	390	210	460	640	540
	Δz (m)	1300	500	500	1300	1300	1300
Rufiji basin (Ruf)	*L* (km)	530	450	240	430	590	530
	Δz (m)	1700	1700	1700	1700	1700	1700
Ruvuma basin (Ruv)	*L* (km)	870	780	600	770	1010	880
	Δz (m)	1700	1700	1700	1700	1700	1700
Lake Nyasa basin (Ny)	*L* (km)	670	610	400	520	700	590
	Δz (m)	0	0	0	0	0	0
Lake Rukwa basin (Ruk)	*L* (km)	490	440	300	320	440	330
	Δz (m)	800	0	0	0	0	0
Lake Tanganyika basin (Tan)	*L* (km)	560	550	480	380	360	320
	Δz (m)	0	800	0	0	0	0
Lake Victoria basin (Vi)	*L* (km)	550	620	720	550	320	420
	Δz (m)	800	800	800	0	0	0

Data and information (Table 4) regarding annual renewable water resources as well as water delivery requirements for irrigation, hydropower generation, water supply, water quality, and livestock for each river basin were collected from previous reports and studies conducted between 1997 and 2006 in the country and were projected to the year 2008 (present situation) by Equations (21) and (22). Although there are many methods of projecting future water delivery requirements, like those suggested by Gleick (1998), there are no sufficient trend data to justify those methods in Tanzania.

4: Total Amount [million cubic meter (mm^3/year)] of Runoff water Generated and Estimated Water Delivery Requirements (in 2008) in each Basin Assessed at 70% Environmental Water Requirements (ewr) of Total Renewable Water Resources. Details of Annual Water Demand in the idb are Given in Table 5

Name of basin	Basin area in Tanzania (km^2)	Renewable water resources (Mm^3/yr)		Total renewable water resources (Mm^3/yr)	Annual water delivery requirements (Mm^3/yr)					Total water delivery requirements	Excess/ deficit of annual renewable water resources (Mm^3/yr)
		Surface runoff	Ground water		EWR	Irrigation	Hydropower	Livestock	Industrial and municipal		
Pangani basin (Pa)	55,000	3,693	7	3,700	2,590	763	2,600	24	113	6,090	-2,390
Wami/Ruvu basin (Wa)	72,930	4,019	9	4,028	2,820	247	none	*	191	3,258	770
Rufiji basin (Ruf)	177,420	30,000	*	30,000	21,000	2,512	4,415	3	74	28,004	1,996
Ruvuma basin (Ruv)	52,200	10,000	*	10,000	7,000	405	none	10	48	7,463	2,537
Lake Nyasa basin (Ny)	37,000	6,510	*	6,510	4,557	31	none	6	31	4,625	1,885
Internal Drainage basin (IDB)	153,800	73	15	88	NA	299	none	108	145	552	-464
Lake Rukwa basin (Ruk)	88,000	4,050	*	4,050	2,835	297	none	5	41	3,178	872
Lake Tanganyika basin (Tan)	151,000	6,900	*	6,900	4,830	142	none	50	109	5,131	1,769
Lake Victoria basin (Vi)	79,570	15,023	*	15,023	10,516	933	none	24	325	11,798	3,225

* No information, Negative values indicate annual deficit in the basin; NA=Not applicable in the IDB.

Several assumptions were made:

i) Water demand, D_t (Mm^3) for water supply and livestock water requirements depend on the population growth rate:

$$D_t = D_0(1+i)^t \tag{21}$$

where,

D_t = Water demand (Mm^3/year) projected using the period of time t (years)

D_0 = Initial water demand (Mm^3/years) of water supply and livestock watering

i = Annual population growth rate (current population growth rate is 2.07%)

t = Time (years)

ii) Water demand for irrigation is given by:

$$I_t = I_0(1 + rt) \qquad (22)$$

where,

I_t = Water demand (Mm^3/year) for irrigation at time t (years)

I_0 = Initial irrigation water demand (Mm^3/years)

r = Annual irrigation growth rate (growth rate is 2.3% per annum)

iii) Negligible changes in the average annual renewable water resources in the source river basins due to climate change.

iv) Water requirements for hydropower generation will remain constant during the design period of the water transfer project to semi-arid region. This is because the new policy in the country puts more emphasis on energy generation from coal, natural gas and wind power.

v) Land is not limited for irrigation expansion since it is estimated that only 15% of all arable land (one million ha) is used for agriculture, and only a fraction is used for irrigation.

Environmental Water Requirements (EWR) were considered as percentage (%) of annual renewable water resources. The estimates for EWR which correspond to “fair” conditions of freshwater-dependent ecosystems worldwide and obtained from simulated hydrological data range from 20% to 50% of the total renewable water resources (Smakhtin et al., 2004). In this study water required for environmental purposes, 5% to 70% of total renewable water per year was considered and was fixed at 70% (Table 4), as beyond 70% some major river basins would move into human water scarcity. The excess water in Table 4 is the difference between the total renewable water and total water delivery requirements per year. This was considered to be the capacity of the basin for water transfer to the IDB. A negative capacity is considered a water deficit for the specific basin. The data (Table 4) shows that Pa basin is under water deficit. However, the excess annual renewable water generated from other basins is sufficient to supply the IDB. Consequently, the Pa basin is not considered in the transportation analysis. Details of annual water delivery requirements for the IDB for the six sub-basins are given in Table 5.

Table 5: Estimated Water Delivery Requirements (in 2008) in Million Cubic Meter Per Year (Mm^3/yr) in the Internal Drainage Basin (IDB)

Sub-basin in semi-arid region	Annual water delivery requirements (Mm^3/yr)			Total annual water delivery requirements (Mm^3/yr)
	Irrigation	Livestock	Industrial and municipal	
Arusha (Ar)	14	20	10	44
Manyara (Ma)	149	20	16	185
Dodoma (Do)	65	30	43	138*
Singida (Si)	0	13	23	36
Shinyanga (Sh)	11	14	35	60
Tabora (Tab)	60	11	18	89
Total	299	108	145	552

*Total water delivery requirements in Dodoma sub-basin is 138 Mm^3. However, annual renewable water generated is 88 Mm^3 (Table 4). An actual water delivery requirement for Dodoma sub-basin from external river basins is only **50 Mm^3**.

5 RESULTS AND DISCUSSION

The objective function [Equation (16)] was minimized by using the *LINGO* package. The contours of optimal costs of water transfer as well as percentage of operation expenses of total expenses are presented in Figure 3 and 4 respectively. The transportation model consists of 27 optimization scenarios according to the following combination:

- Three CRF values: 0.051, 0.066, and 0.084 at the capital interest rates of 4%, 6% and 8% respectively, and 40 years of life span of the pipes.
- Three energy prices (US $/kWh): 0.05, 0.10 and 0.15;
- Three prices of water treatment for poor water quality (US $/$m^3$): 0.05, 0.10 and 0.15.

The results of the first 6 optimization scenarios at CRF of 0.051 and energy price of US $/kWh 0.05 and 0.10 are presented in Table 6. The results show that the capital investments and maintenance costs have high contribution in the total cost of water transfer. There is an insignificant change in total cost when the water treatment cost is increased as demonstrated by optimization scenarios 1 to 3 and 4 to 6. In scenarios 4 to 6 the water treatment price is increased from 0.05 to 0.15 US $/$m^3$ but the total cost of water transfer remains constant at 0.85 US $/$m^3$. This is probably due to the fact that in Equation (11) the parameter α =0.26, namely, only 26% of the water is subject to treatment for industrial and domestic use, while water for irrigation and livestock watering does not need any treatment.

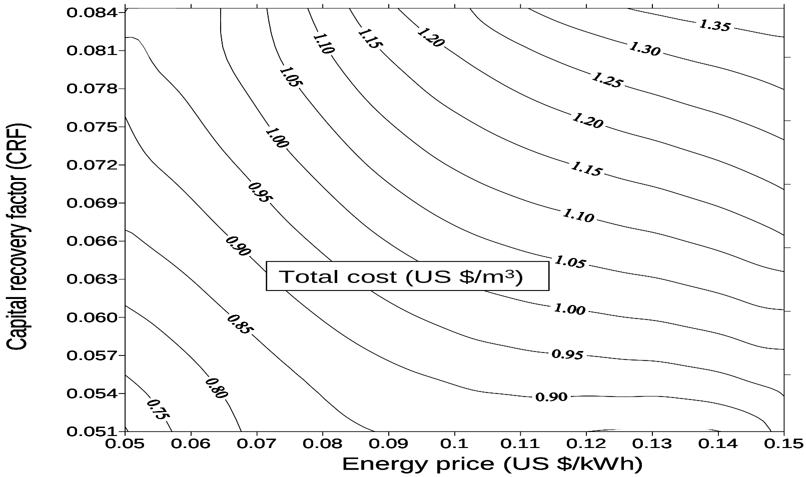
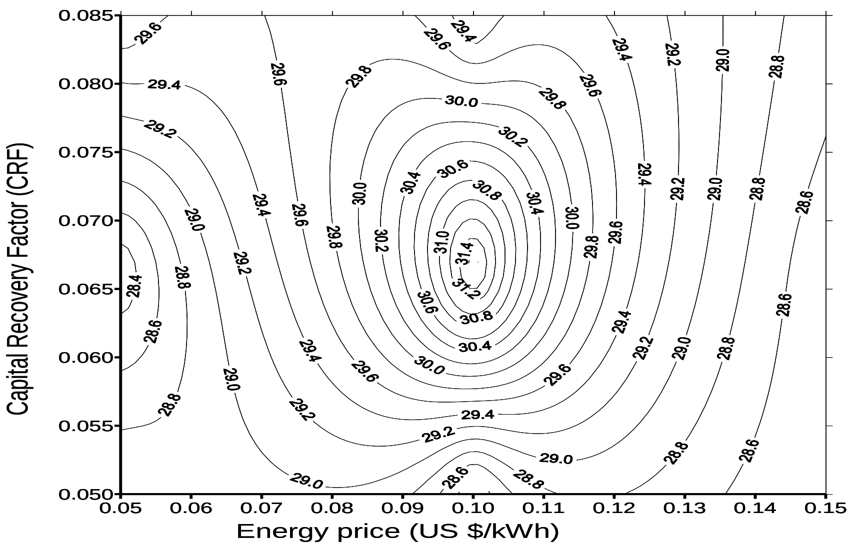


Figure 3: Optimum total costs (US $/m^3) of operation, maintenances and capital investment for water transfer to the Internal Drainage basin (IDB) at different values of CRF and energy prices (Total number of optimization scenarios used was 27)



4: Percentage of operation expenses of total costs of water transfer at different CRF and energy prices, (Total number of optimization scenarios used was 27)

Table 6: Optimization results for the first 6 optimization scenarios conducted at Capital Recovery Factor of 0.051. Total number of optimization scenarios was 27

Optimization Scenario	Optimization running condition		Optimization results		
	Energy price ($/kWh)	Water treatment price ($/m^3)	Operation cost ($/m^3)	Capital investment and maintenance cost ($/m^3)	Total cost of water transfer ($/m^3)
1	0.05	0.05	0.20	0.49	0.69
2	0.05	0.10	0.21	0.49	0.70
3	0.05	0.15	0.22	0.48	0.70
4	0.10	0.05	0.24	0.61	0.85
5	0.10	0.10	0.26	0.59	0.85
6	0.10	0.15	0.26	0.59	0.85

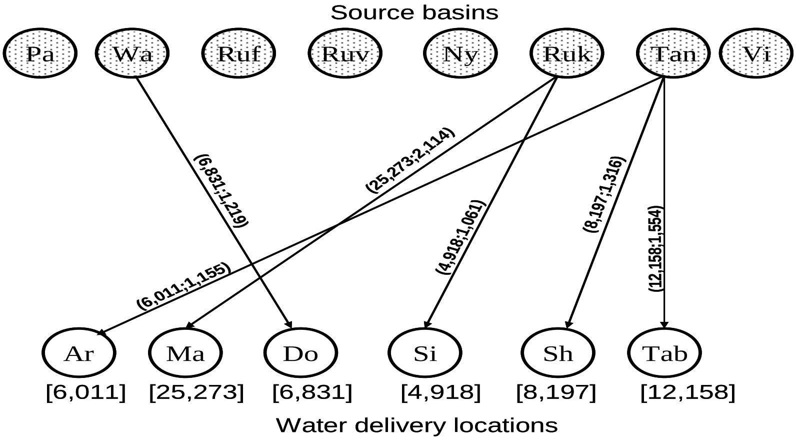
In addition, according to water quality assessment of Table 2, the *WQI* values are between 76 and 98 namely scores of “good” to “excellent” quality that turns the cost of treatment in Equation (11) to be insignificant. Consequently, the sensitivity analysis of total cost refers primarily to energy prices and CRF. Out of 27 optimization scenarios, 11 optimal flow patterns were revealed with different configurations and frequencies of occurrence during the optimization runs. The optimal flow patterns with the corresponding frequencies of occurrence (in brackets) are as follows: P1 (6); P2 (2); P3 (10); P4 (1); P5 (1); P6 (1); P7 (2); P8 (1); P9 (1); P10 (1) and P11 (1). The flow pattern number P3 had the highest frequency of occurrence (10 times). This was followed with P1 that occurred 6 times.

Results of optimum total costs obtained by using 27 optimization scenarios are presented in Figure 3. Optimum total costs lie between 0.69 and 1.37 US $/m^3. The results show that there is a significant increase in the total cost of water transfer when the price of energy or CRF is increased. However, in some parts of the results, for instance when the CRF is less than 0.060, there is a relatively flat shape of contour lines between the energy price of about 0.10 US $/kWh and 0.14 US $/kWh (Figure 3). These are due to changes in the flow patterns and configurations. Increase in energy price means high operation expenses. In this case the model tends to reduce total cost by either increasing pipe diameter or shifting to a different source that is at longer distance than in the former flow pattern, leading to much higher capital cost.

The percentage of operation expenses of total costs of water transfer ranges between 28% and 32% (Figure 4), depending on the CRF and energy price. The other 68% to 72% of the total expenses are for capital investment and maintenances. Results indicate that percentage of operation expenses out of total expenses increases with increase in energy prices until energy price is around US $/kWh 0.10. Beyond this price the percentage of operation costs decreases. This is due to change in the flow patterns that were revealed during optimization runs. The 27 optimization scenarios identified 11 different flow patterns. In order to identify important features of the water transfer system, extreme conditions were examined. These comprised of the flow pattern with minimum and maximum values of total costs of water transfer, as well as that with highest frequency of occurrence.

The same optimum flow pattern number P1 (Figure 5) occurred at minimum total costs of 0.69 US $/m^3 and maximum total cost of 1.37 US $/m^3 of water transfer. Flow pattern number P3 (Figure 6) had the highest frequency of occurrence.

This flow pattern occurred 10 times, with optimum total costs ranging between 0.85 and 1.37 US $/m^3.Results of P1 Figure 5 shows that only three source basins Ruk, Tan and Wa basins are preferred to be used to divert water to IDB region, at optimal total costs ranging between 0.690 US $/m^3 and 1.373 US $/m^3. However, results of P3 (Figure 6) indicate that even two sources (Ruk and Tan basins) can be used to supply the IDB at optimum total costs between 0.848 US $/m^3 and 1.367 US $/m^3. Although the maximum optimum total costs of P1 and P3 are relatively the same, their minima are quite different. Flow pattern number P1 shows that when the costs of operation and capital investments are lower, it can be cost effective subject to three source basins. For high costs of operation and capital investments, P1 and P3 are likely to incur the same costs.



5: Optimum flow pattern number P1 that occurred at minimum and maximum values of total costs of US $/m^3 0.69 and US $/m^3 1.37 respectively. Sources Pa and Vi are not feasible. [The first number in bracket is flow (m^3/h), the second number is optimum diameter of concrete pipe (mm), and the number in square brackets is water delivery requirement (m^3/h)]

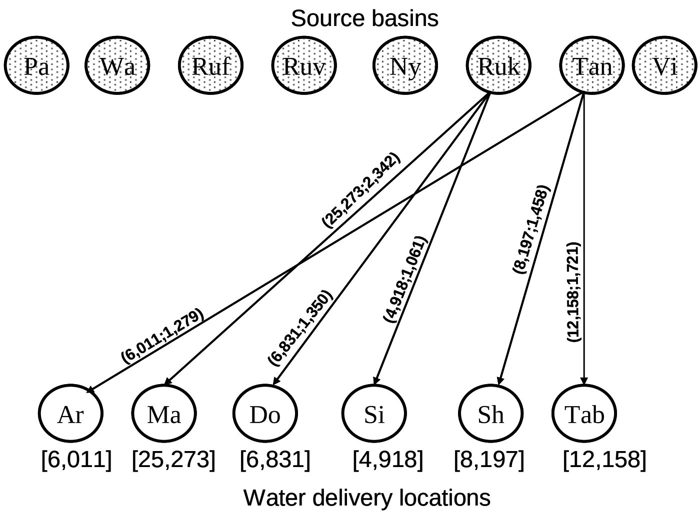


Figure 6: Optimum flow pattern number P3 with maximum frequency of occurrence of 10 out of 27 optimization scenarios. Sources Pa and Vi are not feasible. [The first number in bracket is flow (m^3/h), the second number is optimum diameter of concrete pipe (mm), and the number in square brackets is water delivery requirement (m^3/h)] .

6 CONCLUSIONS

A conceptual management model for optimal trans-regional water transfer is presented, implementing some heuristic principles. The heuristic elements are used for solving the optimization model combined with branch and bound algorithm produces. On the basis of the modeling results, several conclusions can be drawn:

i) Exploitable renewable water resources in Tanzania at present are sufficient to meet the water delivery requirements in Internal Drainage Basin (IDB).

ii) Pangani (Pa) basin is under water shortage. Water withdrawal from Pa basin to supply the semi-arid region would further damage the Pa river ecology, and contribute to growing water conflict among different water users in the basin.

iii) Optimal water allocation shows that Wami/Ruvu (Wa) basin, Lake Rukwa (Ruk) basin and Lake Tanganyika (Tan) basin are reasonably preferred water source for supplying the IDB.

iv) Depending on the capital recovery factor, and energy prices, water may be transferred to the IDB at the total costs that range between 0.69 US $\$/m^3$ and 1.37 US $\$/m^3$ in which the operation costs lie between 28% and 32%. The costs obtained for the trans-region water transfer are relatively high however, in the range of sea water desalination. Implementation of a project of this scale will probably require a governmental support, subject to the national goals.

v) Further research will be conducted towards more practical solution. That will include taking into account the required temporary storage facilities (both volume and location) and considering the seasonal variation of water demands

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