Multi-objective and Risk-based Modelling Methodology for Planning, Design and Operation of Water Supply Systems

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List of Abbreviations

Abbreviation	Denotation
a	annual
DSS	Decision Support System
EIA	Environmental Impact Assessment
EPANET	Simulation program for pressurized networks
FORTRAN	Formula Translator (Programming language)
ft	feet (1 $ft = 0.3048$ m)
GIS	Geographic Information System
gpm	gallon per minute (1 $gpm = 0.000067$ m^3/s)
hr	hour
ILHS	Improved Latin Hypercube Sampling
in	inch (1 $in = 0.0254$ m)
LHS	Latin Hypercube Sampling
m	meter
m.a.s.l	meter above sea level
MCDA	Multiple Criteria Decision Analysis
MCDM	Multiple Criteria Decision Making
MOSA	Multi-objective Simulated Annealing
nmb	number
yEd	Java^{TM} Graph Editor for visualisation of graphs

Notation

Symbol	Definition	Dimension
G	graph	[-]
n_i	node (vertix) i	[-]
N	set of nodes	[-]
a_{ij}	arc (edge) from node n_i to node n_j	[-]
A	set of arcs	[-]
$d(n_i)$	degree of a node n_i	[number]
$do(n_i)$	outdegree of a node n_i	[number]
$di(n_i)$	indegree of a node n_i	[number]
π	path	[-]
π^+	forward path	[-]
π^-	backward path	[-]
G_n	network	[-]
S	arbitrary set	[-]
Q	set of rational numbers	[-]
R_0^+	set of positive rational numbers with zero	[-]
κ_{ij}	upper capacity of arc a_{ij}	[m^3/s]
λ_{ij}	lower capacity of arc a_{ij}	[m^3/s]
x_{ij}	flow of water in arc a_{ij}	[m^3/s]
$\mathbf{x}$	flow vector (flow pattern) on a network G_n	[m^3/s]
x^π	flow on path π (path flow)	[m^3/s]
L_{ij}	length of pipe a_{ij}	[m]
A_{ij}	cross section area of pipe a_{ij}	[m^2]
$C_{\tau_{ij}}$	friction coefficient of pipe a_{ij}	[number]
r_{ij}	pipe characteristics of a_{ij}	[number]
z_i	elevation of node n_i	[m]
p_i	hydrostatic pressure at node n_i	[Pa]
Π_i	pressure head at node n_i	[m]
b_i	external flow at node n_i	[m^3/s]
S_{max_i}	maximum available supply at node n_i	[m^3/s]
D_{min_i}	minimum needed demand at node n_i	[m^3/s]
λ_{ij}	Darcy-Weissbach friction coefficient of pipe a_{ij}	[number]
C_{ij}	Hazen-Williams friction coefficient of pipe a_{ij}	[number]

n_{ij}	Chezy-Manning friction coefficient of pipe a_{ij}	[number]
k	unit conversion factor bertween English and SI units	[number]
z	total costs of the network (flow transport) problem	[value]
c_{ij}	cost coefficient of arc a_{ij}	[$value/[x]$]
π_k	conforming simple path	[-]
x^{π_k}	flow on conforming path π_k	[m^3/s]
$c(x)$	cost function of some system parameter x	[$value/[x]$]
q, p, r	parameters of some cost function $c(x)$	[number]
$s(c)$	scaling function of some cost function $c(x)$	[value]
$C(x)$	unit cost function of some cost function $c(x)$	[number]
$C^l(x)$	total impact function of some system parameter x	[number]
$C^{ecn}(x)$	economic impact function of some system parameter x	[number]
$C^{env}(x)$	environmental impact function of some system parameter x	[number]
$C^{soc}(x)$	socio impact function of some system parameter x	[number]
$C^{syst}(x)$	system quality impact function of some system parameter x	[number]
z^i	solution of the network flow problem on objective i	[value]
z^i_w	solution of the network flow problem on objective i for combination of weights toward different objectives w	[value]
$C_{fix_{ij}}$	fixed impact function for some system element a_{ij}	[number]
$C_{var_{ij}}$	variable impact function for some system element a_{ij}	[number]
PV	present value of some investment	[value]
FV	future value of some investment	[value]
A	annuity for some system costs or benefits in some compounding period	[value]
a_t	weights toward different annuity values	[value]
PVA	present value to annuity of some investment	[value]
FVA	future value to annuity of some investment	[value]
DPV	discounted present value of some investment	[value]
r	interest rate	[%]
t	time period	[years]
n	length of time period	[years]
DC_{var}	discounted variable impacts to the present value	[value]
y_{ij}	integer variable to distinguisch existing and potential elements a_{ij}	[number]
O	set of origin nodes	[-]
T	set of terminal nodes	[-]
$A^{O,T}$	cut in a network G_n with sets O and T	[-]
$\kappa^{O,T}$	cut in a network G_n with sets O and T	[-]
π_a	augmenting path	[-]
x^{π_a}	flow on augmenting path π_k	[m^3/s]
$\mathbf{X}$	set of flow vectors on a network G_n	[m^3/s]
Z	set of function values (solutions) for a set of flow vectors $\mathbf{X}$	[value]
T	temperature at some energy level in Simulated Annealing	[number]
T_{max}	initial temperature in Simulated Annealing	[number]
T_{min}	minimal temperature in Simulated Annealing	[number]

ΔT	temperature decrease parameter in Simulated Annealing	[number]
N_{max}	maximal number of changes at some energy level in Simulated Annealing	[number]
N_{succ}	maximal number of successful changes at some energy level in Simulated Annealing	[number]
B	constant that relates temperature to the function value in Simulated Annealing	[number]
Δx	random flow change in Simulated Annealing	[m^3/s]
Δz	change of the function value (solution) in Simulated Annealing	[value]
ΔP	penalty constant for the consideration of pressure constraints	[value]
W^l	combination of weights toward different function criteria l	[number]
Ω	set of combination of weights W^l	[number]
Ω^D	set of dominant combination of weights W^l	[number]
Δz^l	change of the function value on criteria l in MOSA	[value]
Δz_w	weighted change of the function value for all criteria in MOSA	[value]
Δs	aggregate function change in MOSA	[number]
$O()$	time complexity function	[number]
$\underline{z}$	lower bound solution in Branch and Bound	[value]
$\overline{z}$	upper bound solution in Branch and Bound	[value]
s	failure scenario in Path Restoration Method	[-]
f	failure source-destination paths for scenario s	[-]
F	set of all failure source-destination paths for failure scenario s	[-]
Q^s_f	total affected flow on failed path f for failure scenario s	[m^3/s]
r	restoration source-destination paths for failed path f	[-]
R	set of all restoration source-destination paths for failed path f	[-]
${x^{\pi}_{f,r}}^s$	flow on restoration path r for the failed path f in failure scenario s	[m^3/s]
$H_s({x^{\pi}_{f,r}}^s)$	head at source node of the path ${x^{\pi}_{f,r}}^s$	[m]
$H_d({x^{\pi}_{f,r}}^s)$	head at destination node of the path ${x^{\pi}_{f,r}}^s$	[m]
$\Delta H(\delta^s_{f,r_i}{x^{\pi}_{f,r}}^s)$	sum of all losses on the path ${x^{\pi}_{f,r}}^s$	[m]
D_i	variable i in sampling technique ILHS	[-]
N	number of variables to sample with ILHS	[number]
j	interval of the sampling in ILHS	[-]
M	number of sampling intervals in ILHS	[number]
$P(D^j_i)$	probability density function of the interval j in variable i	[number]

Subscripts:		**Superscripts:**	
i	node	π	path
s	supply node	l	objective, criteria
d	demand node	$'$	iteration
t	transshipment node	$''$	next iteration
S	slack node	$\dot{}$	temperature level
ij	arc	$\ddot{}$	next temperature level
k	conforming	D	dominant solutions
a	augmenting	s	failure scenario
w	weighted	t	time
fix	fixed	env	environmental
var	variable	ecn	economic
min	minimum	soc	socio
max	maximum	$syst$	system quality

Abstract

The ongoing changes in the society's perception of the role and function of infrastructure systems as well as degradation of the state of natural resources, increasingly appoint new challenges to the management of water supply systems. Out of many, the main research objectives of this research are: the integration of multiple objectives and criteria, and the incorporation of uncertainty, risk and reliability considerations in the water supply systems analysis. In order to help to implement these objectives in everyday planning, design and operation of water supply systems, an unique optimisation methodology has been developed and implemented into corresponding computer models.

The methodology uses the network approach for conceptual and structural representation of water supply systems and define planning, design and operation management problems as Network Minimum Cost Flow problems with multiple objectives. Different impacts of water supply projects or actions such as economic costs, environmental consequence or social disapproval are add together according to the utilities (preferences) of decision makers by implementing the Multi Objective Simulated Annealing (MOSA) method. In order to improve the performance of the algorithm for complex combinatorial problems and reduce questioning of non-optimal alternatives, the MOSA algorithm is embedded into the Branch and Bound method. For optimisation problems defined on networks, the combination of the previous two algorithms provide for robust and efficient identification of Pareto-solutions.

The inclusion of uncertainty, risk and reliability considerations in the analysis is based on the Stochastic design approach. It provides for the inclusion of decision makers' risk perception in evaluation of the satisfactory system's performance. The accepted risk for some system configuration is obtained as a statistical expectation of the costs of expected failures. A deterministically defined failure of an individual system component is considered with an advanced Path Restoration method, while a probabilistically defined performance failure is addressed with stochastical simulation of system's performances. An advanced sampling method (i.e. Latin Hypercube) is used for the creation of representative samples of uncertain and variable parameters. The system's reliability is obtained form the statistical analysis of calculated system's performances evaluated with predefined risk tolerance levels.

Finally, a demonstration at a) a multi-objective planning problem of a system expansion, b) a NP-hard design problem of pipe diameters selection and c) a complex operation problem of pump scheduling is done on the basis of well known test studies from the literature. These proved that network system representation, multi-objective problem formulation and inclusion of decision makers' preferences and risk perception in the development of optimal alternatives improve the creation of Pareto-optimal solutions, increase the efficiency of optimisation procedure and add to the transparency of the system analyse.

Zusammenfassung

Motivation und Zielsetzung

Die verstärkte Nutzung der natürlichen Wasserressourcen und die weltweite Verunreinigung dieses kostbaren Schatzes im 20. Jahrhundert führte zur Erschöpfung und Verschmutzung vieler natürlicher Wasserkörper und zur Zerstörung zahlreicher Ökosysteme. Die wachsende Spannung zwischen intensiver Wassernutzung und der natürlichen Funktion von Ökosystemen, veränderte unsere Vorstellung über die Aufgabe der Wasserversorgungssysteme von den *human utility services* hin zu den *coupled human-natural systems* (Allenby, 2004). Die integrative Betrachtung von Umwelt und künstlichen Systemen, stellt ein neues Paradigma unserer Gesellschaft dar (IUCN et al., 1980; UN, 1992). Allerdings stehlt die integrierte Betrachtung von gesellschaftlicher, ökonomischer und ökologischer Aspekte von Wasserversorgungssystemen eine große Herausforderung dar, nicht nur aufgrund unterschiedlicher zeitlicher, räumlicher Wertmaßeinheiten und Skalen dieser verschiedenen Aspekte, sondern auch wegen ihres sehr unsicheren und empfindlichen Charakters. Aus diesem Grund bildet, die Notwendigkeit für die integrative Analyse aller dieser Aspekte den Hauptbeweggrund dieser Studie.

Modernes Management der Wasserversorgungssysteme basiert nicht nur auf der Anwendung der besten verfügbaren technischen Maßnahmen, sondern auch auf der Nutzung fortgeschrittener Rechenmodelle für die Auswertung, Analyse, Steuerung, Betrieb und Entwicklung der Systeme. Optimale Alternativen unter Berücksichtigung von bestimmten Managementzielsetzungen und Entscheidungstrefferpräferenzen können nur mit Hilfe von Entscheidungsunterstützungssystemen entwickelt und festgelegt werden. Deshalb bildet die Entwicklung einer systematischen Methodologie und der dazugehörigen Werkzeuge für eine bessere Entscheidungsunterstützung im Management der Wasserversorgungssysteme, den Hauptfokus dieser Arbeit.

Um zwischen den vielfältigen Tätigkeiten im Rahmen des Managements von Wasserversorgungssystemen unterscheiden zu können, wird häufig der von Jamieson (1981) entwickelte Ansatz verwendet. Dieser unterscheidet Planungs-, Entwurfs- und Betriebsstadium (Abbildung 0.1). Außerdem wird die Systemanalyse hier als "Suchverfahren, um ein System zu optimieren" gesehen, wo: a) die Planung sich auf die Entwicklung der Systemstruktur konzentriert, b) der Entwurf optimale Systemkomponenten definiert, um die erforderte Systemleistungen zu erfüllen und c) im Betriebsstadium Haltungskosten optimiert, Instandhaltungsstrategien entwickelt und Systemsleistungen verbessert werden.

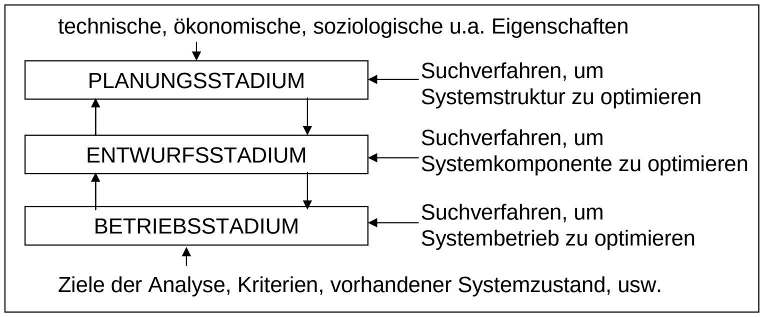


Abb. 0.1.: Hierarchischen Ansatz zu Wasserversorgungsmanagement (Jamieson, 1981)

Erläuterung der Methodologie

Die in dieser Arbeit entwickelte Methodologie verwendet den Netzwerkansatz für die konzeptionelle und strukturelle Darstellung der Wasserversorgungssysteme und definiert damit ein *Network Minimum Cost Flow Problem* mit mehrfachen Zielsetzungen, um Planungs-, Entwurfs- und Betriebsmanagementprobleme mathematisch zu formulieren. Unterschiedliche Aspekte von Wasserversorgungsprojekten und -aufgaben, wie Minimierung von ökonomischen Kosten, Umweltauswirkungen oder negativen soziale Folgerungen, werden den Präferenzen von Entscheidungsträgern entsprechend, mit der *Multi-objective Simulated Annealing* (MOSA) Methode (Ulungu et al., 1995; Kirkpatrick et al., 1983; Cerny, 1985) zusammengeführt. Um die Leistungsfähigkeit des Algorithmus für komplizierte kombinatorische Probleme zu verbessern und das Abfragen der nicht-optimalen Alternativen zu verringern, wird der MOSA Algorithmus in die *Branch and Bound* Methode (Land 1960) eingebettet. Für gut strukturierte Netzwerk-Optimierungsprobleme gewährleistet die Kombination der beiden genannten Algorithmen eine robuste und leistungsfähige Kennzeichnung der Pareto-optimalen Lösungen (Lösungen, bei denen die Verbesserung eines Kriteriums nicht erzielt werden kann, ohne eine Verschlechterung bei mindestens einem anderem zu verursachen).

Eine methodische Einbeziehung der Unsicherheiten und der Veränderlichkeit der Eingangsparameter wird erreicht, indem man unterschiedliche mögliche Systemalternativen mit Hilfe der stochastischen Simulationsverfahren evaluiert. Die dafür nötigen repräsentativen Stichproben der Eingangsparameter wurden mit der *Latin Hypercube Sampling* Technik (Iman and Shortencarier 1984) generiert. Eine statistische Analyse der berechneten Systemsleistungen für diese Stichproben wird dann für die Einschätzung der Systemzuverlässigkeit verwendet. Zusammen mit der Ausfallanalyse, welche durch das *Pat Restoration* Verfahren (Iraschko et al. 1998) eingeführt worden ist, wird die Kompromissfindung zwischen der Systemzuverlässigkeit und Kriterien wie ökonomische Kosten ermöglicht. Da ein solcher Kompromiss stark von den einzelnen Präferenzen oder Risikoeinschätzungen der Entscheidungsträger oder der Systembenutzer abhängig ist, wurden diese bereits in die Formulierung des Problems einbezogen. Die Präferenzen der Entscheidungsträger bei einzelnen Zielkriterien im Planungsstadium werden durch Gewichte ausgedrückt. Für das Entwurfsstadium, ist der stochastische Ansatz (*Stochastic Design*) von Henley and Kumamoto (1981); Ang and Tang (1984); Plate (2000) herangezogen worden, der die risikoorientierte Definition der Systemleistungen und die Konsequenzen eines Versagens als statistische Erwartung aller erwarteten Ausfälle ausdrückt.

Die beschriebene Methodologie wurde in drei entsprechenden Computermodellen umgesetzt. Sie sind an die spezifischen Aspekte der Wasserversorgungsplanung, des Entwurfes und des Betriebsmanagements angepasst und ermöglichen im Verbund eine volle Entscheidungsunterstützung im Management von Wasserversorgungssystemen.

Erläuterung der Modellen

Die Struktur der drei Teilmodelle und die durch sie berechneten Ergebnisse werden in dieser Übersicht prinzipiell angand von Fallstudien erlautert, die in der Arbeit im Detail dargestellt und anusgewartet werden. Dieses Vorgeben scheint am besten geeignet, um einen rascher Überblick über die Zielsetzung, die Vergehensweise, die Leistungen und den Anwendungsnutzen der Modelle zu geben

Das Planungsmodell

Die erste ausgewählte Fallstudie ist die so genannte "2-loop" Studie von Alperovits and Shamir (1977). Sie stellt ein Standardproblem für die Dimensionierung von Wasserverteilungsnetzen dar und wurde für die Planungszwecke hier etwas modifiziert. Das originale Netz (innerhalb der punktierten Linie in Abbildung 0.2) besteht aus 8 Wasserleitungen (dargestellt als Pfeile), 6 Bedarfspunkten (dargestellt als Paralleltrapeze) und einer einzelnen Flusswasserentnahme *N1* (dargestellt als Ellipsoid). Für das Planungsproblem der Entwicklung von neuen Wasserversorgungsstrategien in den nächsten Jahren, sind neben schon existierenden Wasserversorgungskomponenten, drei neue mögliche Wasserentnahmestellen (Quellen *N8* und *N9*, sowie Grundwasserbrunnen *N10*) mit drei entsprechende Transportleitungen (*A9*, *A10* und *A11*) in Betracht gezogen worden.

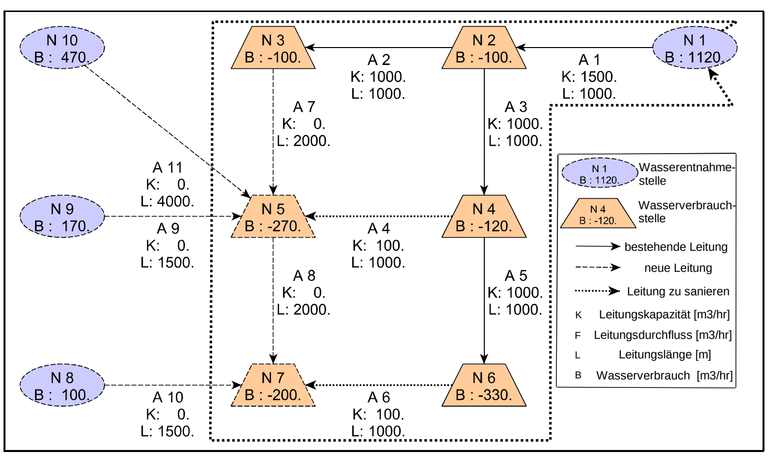
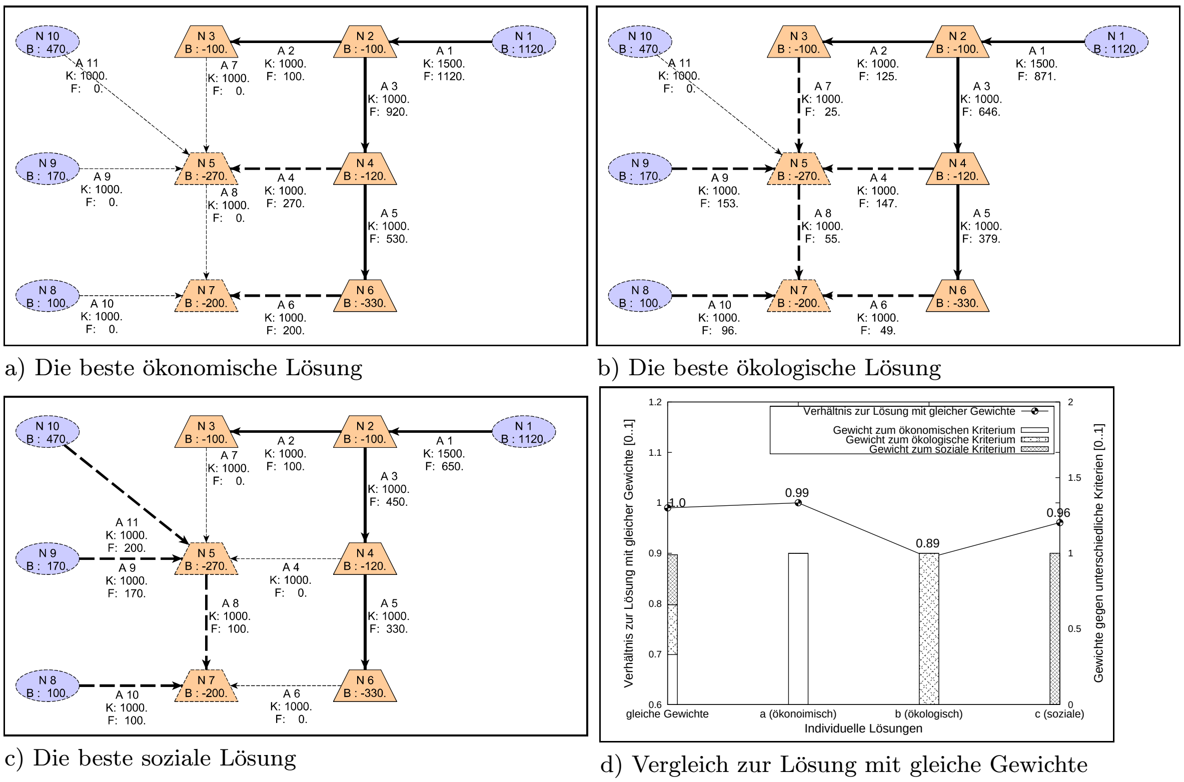


Abb. 0.2.: Fallstudie $P1$: Netzkonfiguration [Adaptation von Alperovits and Shamir (1977)]

Das zu lösende Problem ist die Bestimmung der Entnahmestellen oder Kombination von Entnahmestellen mit den entsprechenden Transportoptionen, um eine "optimale" Wasserversorgung bezüglich des voraussichtlichen Wasserverbrauchs in der Planungsperiode zu gewährleisten. Das "Optimum" wird hier durch drei Hauptzielsetzungen definiert: 1) Senkung ökonomischer Kosten, 2) Minimierung der Umweltauswirkungen und 3) Vermeidung sozialer Belastungen. Obwohl das Wasser aus der bereits vorhandenen Flusswasserentnahme *N1* sehr kostengünstig transportiert werden kann, haben große Entnahmen negative Auswirkungen für das Flussökosystem. Andererseits ermöglichen das Quell- und Grundwasser (*N8, N9, N10*) eine bessere Verteilung der Umweltbelastung, sind aber mit großen Investitionskosten verbunden. Zusätzlich wird das Grundwasser als strategische Wasserressource angesehen und große Entnahmewerte können negative soziale Folgen haben.

Individuelle Lösungen - Vor der Entwicklung von Mehrziel-Lösungen ist es häufig ratsam, die optimalen Lösungen für jedes separate Kriterium zu bestimmen. Diese Lösungen bilden die Grenzen des Lösungsraumes und stellen die extremen Anlagenkonfigurationen dar, die nur eine Zielsetzung bevorzugen (Abbildung 0.3).



a) Die beste ökonomische Lösung

b) Die beste ökologische Lösung

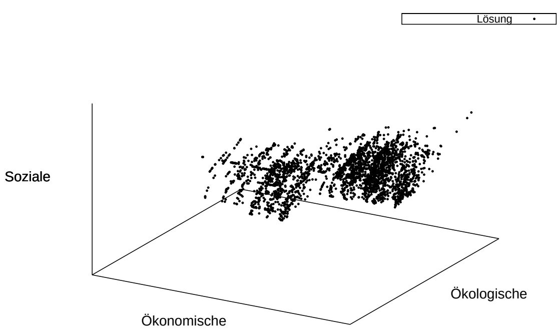
c) Die beste soziale Lösung

d) Vergleich zur Lösung mit gleiche Gewichte

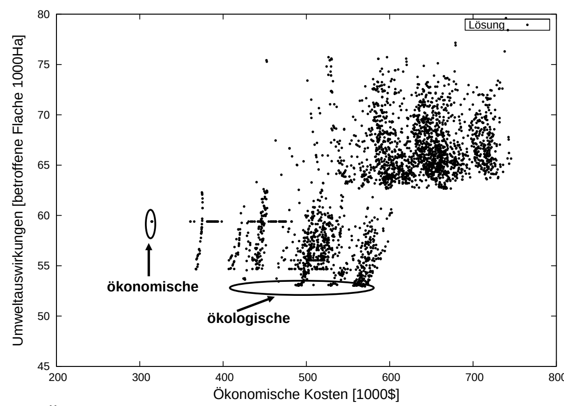
Abb. 0.3.: Fallstudie $P1$: Berechnete individuelle Lösungen

Wie erwartet, schlägt die ausschließlich ökonomisch orientierte Lösung (Diagramm a in Abbildung 0.3) die Rehabilitation und Nutzung der vorhandenen Leitungen *A4* und *A6* als die optimale Wahl vor. Die Summe der Investitions- und Betriebskosten ist für diese Systemkonfiguration deutlich niedriger, als für die Einschließung neuer Entnahmestellen. Obgleich die hohe Nutzung von Flusswasser ($F_{N1} = 1120\ m^3/day$) große negative Umwelt- und soziale Konsequenzen hervorruft, werden diese zwei Aspekte in dieser Lösung vernachlässigt. Andererseits schlägt die optimale umweltorientierte Lösung (Diagramm b in Abbildung 0.3) den Gebrauch von Quellwasser ($F_{N9} = 151\ m^3/day$ und $F_{N8} = 97\ m^3/day$) als die optimale Wasserversorgungsvariante vor, da für diese zwei Entnahmestelle sehr geringe Umweltauswirkungen angenommen worden sind. Im Gegensatz dazu verteilt die optimale Lösung für eine Minimierung der sozialen Folgen (Diagramm c in Abbildung 0.3) die Wasserentnahme auf alle Wasserentnahmestellen gleichmäßig ($F_{N10} = 200\ m^3/day$, $F_{N9} = 170\ m^3/day$ und $F_{N8} = 100\ m^3/day$).

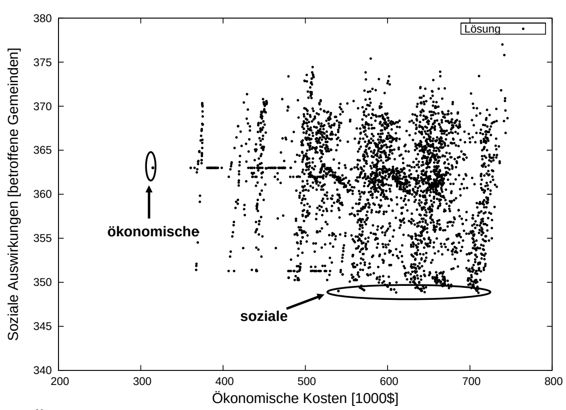
Mehrziel Lösungen - Bei der Berechnung von optimalen Lösungen für alle drei Ziele bestimmen die Werte der unterschiedliche Kriterien eine Punktwolke (Diagramm a in Abbildung 0.4). Offensichtlich gibt es statt einer einzelnen Lösung, welche bezüglich aller Kriterien die optimale ist, viele äquivalente Lösungen, die zu unterschiedliche Präferenzen im Bezug auf verschiedene Kriterien am besten passen. Solche optimale Lösungen befinden sich am Rand der Lösungswolke und formen optimale Lösungssätze (Diagramme b, c, und d in Abbildung 0.4).



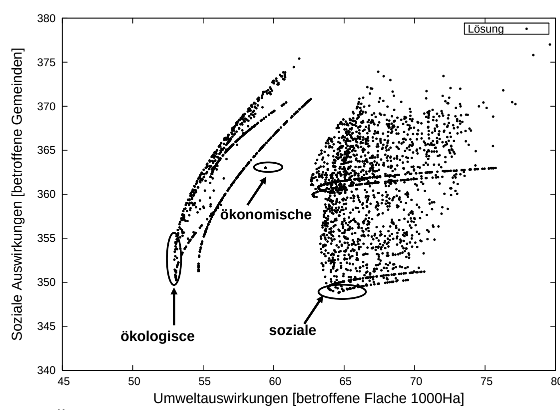
a) Ökonomische, ökologische und soziale Kriterien



b) Ökonomische gegen ökologische Kriterien



c) Ökonomische gegen soziale Kriterien



d) Ökologische gegen soziale Kriterien

Abb. 0.4.: Fallstudie $P1$: Berechnete Werte der ökonomischen, ökologischen und sozialen Kriterien für die Mehrziel-Lösungen

Die Gruppierung der Lösungen und die diskontinuierlichen optimalen Lösungssätze, besonders in Hinsicht auf die ökonomische Kriterien, sind als Folge der diskontinuierlichen Systemstruktur zu erklären. Solche Problemdefinitionen, bei denen zwischen "ja" und "nein" oder "zu bauen" und "nicht zu bauen" auszuwählen ist, fördert nur eine kostengünstigste Lösung (Diagramme b und c in Abbildung 0.4). Diese Lösung ist aber sehr schlecht im Hinblick auf ökonomische und soziale Kriterien (Diagramm d in Abbildung 0.4). Anderseits es ist festzustellen dass im Bezug auf ökologische und sozial Kriterien mehrere gleich gute Lösungen für unterschiedliche ökonomische Kosten identifiziert werden können.

Das Entwurfsmodell

Die Entwurfsanalyse ist eine Fortsetzung der Planungsanalyse, in der die Kapazitäten der Netzelemente für die ausgewählte optimale Netzkonfiguration festgestellt werden sollen. Diejenige Netzkonfiguration, die im Bezug auf soziale Kriterien optimal ist und alle drei neue Wasserentnahmestellen (*N8, N9 und N10*) bevorzugt, wird hier vom Entwurfsstandpunkt aus optimiert. Zu optimieren sind die Netzdurchflüsse und -durchmesser die minimale Investitions- und Betriebskosten haben aber trotzdem ein bestimmten Niveau von Systemzuverlässigkeit gewährleisten. Die Systemzuverlässigkeit wurde durch Analysieren von Systemverhalten unter Berücksichtigung von mögliche Ausfälle der individuelle Systemkomponenten und unter Berücksichtigung von Unsicherheiten in den Eingabeparametern (z.B. Wasserbedarf) bestimmt.

Die kostengünstigste Lösung wurde zuerst berechnet. Die standardmäßigen Leitungsdurchmesser und die entsprechenden Investitions- und Betriebskosten wurden aus der Studie von Alperovits and Shamir (1977) entnommen. Die berechneten optimalen Netzdurchflüsse und die entsprechenden kostengünstigsten Netzdurchmesser, die für den Transport der erforderlichen Wassermengen und einen minimalen Druck von 30 *m Wassersäule* an jedem Bedarfspunkt erforderlich sind, sind in Tabelle 0.2 dargestellt. Diese Ergebnisse stimmen mit denen anderer Studien, die sich mit dem gleichen Problem befasst haben und andere Optimierungsverfahren verwendet haben z.B. *Genetic Algorithm* (Savić and Walters, 1997), *Search Algorithm* (Abebe and Solomatine, 1998), *Simulated Annealing* (Cunha and Sousa, 1999), *Shuffled Frog Leaping* (Eusuff and Lansey, 2003) und *Shuffled Complex Evolution* (Liong and Atiquzzaman, 2004) überein.

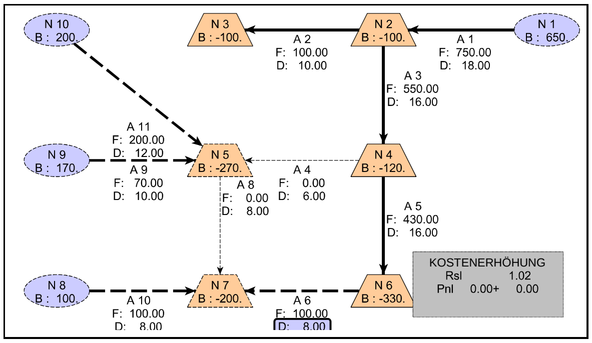
Rohre	Länge [m]	Reibungskoeffizient	Durchfluss [m3/day]	Druckverlust [m]	Durchmesser [inch]	Durchmesser [m]
1	1000	130	650.00	2.46	18	457.2
2	1000	130	100.00	1.35	10	254.0
3	1000	130	450.00	2.21	16	406.4
4	1000	130	0.00	0.00	6	152.4
5	1000	130	330.00	1.25	16	406.4
6	1000	130	0.00	0.00	6	152.4
8	2000	130	100.00	7.99	8	203.2
9	1500	130	170.00	5.40	10	254.0
10	1500	130	100.00	5.99	8	203.2
11	4000	130	200.00	8.00	12	304.8

Knoten	Geodet. Höhe [m]	Energie Höhe [m]	Druck [m]
1	150	180.00	30.00
2	120	177.54	57.54
3	130	176.19	46.19
4	125	175.33	50.33
5	120	175.33	55.33
6	135	174.08	39.08
7	130	167.34	37.34
8	150	180.00	30.00
9	150	180.00	30.00
10	150	180.00	30.00

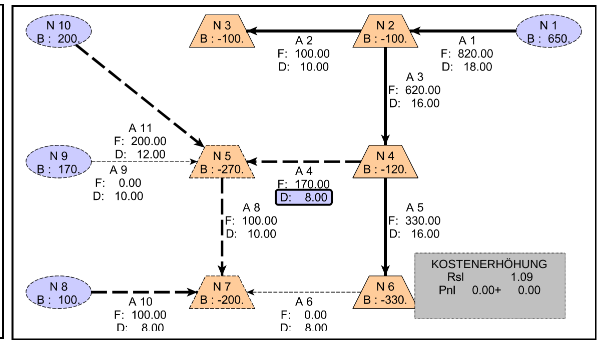
Tab. 0.2.: Fallstudie $D1$: Berechnete optimale Durchflusse, Druck und Druckverluste

Obwohl optimal angesichts der ökonomischen Kosten, bietet diese Lösung sehr wenig Zuverlässigkeit und Sicherheit im Betrieb und ist von geringem praktischen Wert. Deshalb wird dieser Ein-Kriterium Entwurfsansatz um eine Ausfall- und Unsicherheitsanalyse erweitert.

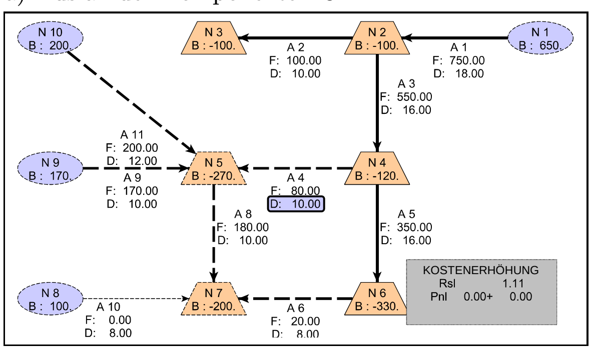
Ausfallanalyse - Die Entwurfsanalyse muss in der Lage sein, eine Reihe von Betriebszuständen anzusprechen, wobei der Ausfall eines beliebigen Netzbestandteils ein Standardproblem darstellt. Die hier verwendete Methode für die systematische Erweiterungen der Systemkapazität basiert auf der *Path Restoreation* Methode von Iraschko and Grover (2000). Die Ergebnisse der Ausfallanalysen für alle Leitungen (*A8, A9, A10, A11*) die das Wasser zu den Verbrauchern *N5 und N7* liefern und die resultierenden Zunahmen der Netzdurchmesser werden in Abbildung 0.5 dargestellt.



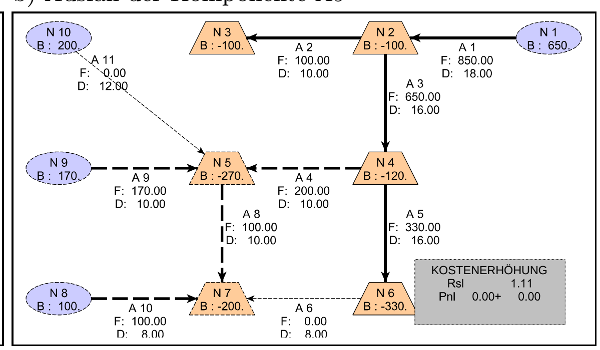
a) Ausfall der Komponente $A8$



b) Ausfall der Komponente $A9$



c) Ausfall der Komponente $A10$



d) Ausfall der Komponente $A11$

Abb. 0.5.: Fallstudie $D1$: Berechnete optimale Erweiterung von Leitungsdurchmessern für Ausfälle der Komponenten *A8, A9, A10, A11*

Es wird deutlich, dass die Erweiterung der Kapazität an den Leitungen $A4$ und $A6$ mit einer Erhöhung der Gesamtkosten um 11 %, volle Systemleistungen beim Ausfall von je einer der vier Leitungen *A8, A9, A10, A11* ermöglicht. Da die Leitungen $A4$ und $A6$ die niedrigsten Investitionskosten haben, war zu erwarten, dass das Entwurfsmodell genau diese Leitungen erweitert.

Unsicherheitsanalyse - Weiterhin soll die Entwurfsanalyse die Frage der variablen und unsicheren Entwurfsparameter betrachten. Statt ein System, das den gesamten Unsicherheitsraum abdeckt zu entwerfen, ein Ansatz, der das Leistungspotential beliebiges Systementwurfes für probabilistisch definierte unsichere Parameter berechnet, wurde hier verwendet. Dieses Potential definiert im wesentlichen die Systemzuverlässigkeit und wird iterativ für die Erkennung und Bestimmung von weiteren Verbesserungen der vorgeschlagenen Systemlösungen verwendet, abhangig von der Risikobereitscheft und -akzeptanz der Entscheidungsträger.

Für die Abbildung des Entwurfsmodells wird der Wasserbedarf an allen Bedarfspunkten des oben genannten Problems als unsicherer Eingabeparameter mit gleichwertiger Wahrscheinlichkeitsdichtefunktion und angenommener Abweichung vom Mittelwert von 25 % betrachtet. Die Zuverlässigkeit des Systems wurde a) unter der Annahme der Unabhängigkeit des Wasserbedarfes am verschiedenen Bedarfspunkten und b) mit einer angenommenen Abhängigkeit des Wasserbedarfs ausgewertet. Die *Latin Hypercube Sampling* (Iman and Shortencarier, 1984) Methode wurde verwendet, um Stichproben zu erstellen. Die Leistung des Netzes für beide Proben wurde dann mit dem Netzsimulator von Gessler et al. (1985) errechnet. Die statistische Auswertungen des berechneten Drucks werden in Abbildung 0.6 gezeigt.

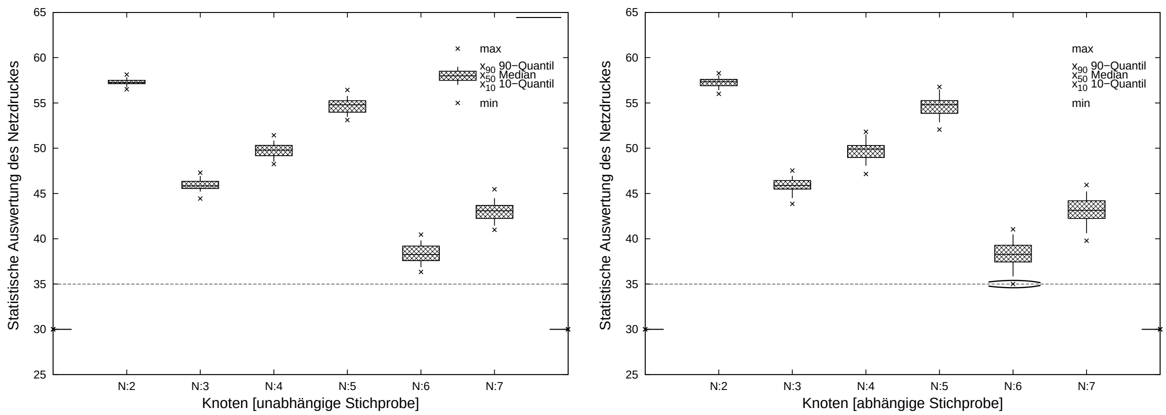


Abb. 0.6.: Fallstudie $D1$: Statistische Auswertung von berechneten Drücke für Stichprobe ohne und mit Bedarfsbeziehung

Beide Diagramme zeigen ein sehr zuverlässiges Systemsverhalten im Hinblick auf minimale und maximale errechnete Drücke im Netz. Das ist mit der Erweiterung von Netzkapazitäten in der Ausfallanalyse zu erklären. Es ist auch festzustellen, dass die angenommene Bedarfsbeziehung die Leistungen des Systems beeinflusst, indem der minimale berechnete Druck etwas niedriger ist (der Druck am Bedarfspunkt *N6* dargestellt im rechten Diagramm erreicht den Grenzwert von 35 *m Wassersäule*). Trotzdem ist die Wahrscheinlichkeit des Auftretens solcher Drücke kleiner als 10 %.

Um die Systemzuverlässigkeit und das Systemverhalten, unter Annahme von unterschiedlichen anderen Eingabeparametern mit verschiedenen Wahrscheinlichkeitsdichtefunktionen und Abweichungsbereichen einzuschätzen, kann die Unsicherheitsanalyse mit diesen Eingaben wiederholt werden. Die Ergebnisse könnten dann entsprechend den Präferenzen und Risikobereitschaft der Entscheidungsträger evaluiert werden, um die optimale Lösung für einen bestimmten akzeptablen Grad der Unsicherheit zu identifizieren.

Das Betriebsmodell

Im Gegensatz zur Entwurfsoptimierung, die auf einer stationären Systemsimulation basiert, betrachtet die Betriebanalyse das dynamische Systemverhalten, um die Dimensionierung von Behältern und Speichern sowie die Entwicklung der Betriebregeln für die Pumpen, Ventile, Niveaukontroller und Messgeräte zu ermöglichen. Für die Darstellung von hier entwickelten Betriebsmodell, wird wieder die Fallstudie von Alperovits and Shamir (1977) verwendet. Da das originale Problem nicht die Betriebskosten einbezieht, werden diese nach der Fallstudie von Walski et al. (1987) eingebaut. Zusätzlich ist eine Pumpe (im Punkt *N1*) und ein Wasserbehälter (im Punkt *N12* an Punkt *N2* angeschlossen) hinzugefügt. Die im Entwurfsstadium berechneten Leitungsdurchmesser werden hier als gegeben angenommen. Schließlich werden die täglichen Schwankungen der Wassernachfrage, wie in Walski et al. (1987), und die täglichen Veränderungen der Energiekosten durch ein 3-Phasesystem mit den Koeffizienten 1.0 ("normal"), 1.5 ("hoch") und 1.2 ("niedrig") einbezogen.

Optimierung der Pumpensteuerung - Aus betrieblicher Sicht ist die Pumpenbetrieb oft das kostenintensivste Element eines Wasserversorgungssystems. Deshalb wurde die Erstellung der kostengünstigsten Pumpensteuerung für eine gegebene tägliche Wasserbedarfschwankung mit den vorhandenen Kapazitäten der Wasserbehältern als erster Schritt des Betriebsoptimierungsverfahrens ausgewählt. Das hier angewendete Optimierungsverfahren für eine Simulation über 24 h mit einem angenommenen Tankquerschnitt von 50 m^2 und einem minimalen und maximalen Wasserniveau im Behälter von 15 bzw. 60 m, berechnete eine optimale Pumpensteuerung wie im linken Diagramm in Abbildung 0.7 dargestellt. Das entsprechende Wasserniveau im Behälter $N12$ ist im rechten Diagramm in der gleichen Abbildung dargestellt.

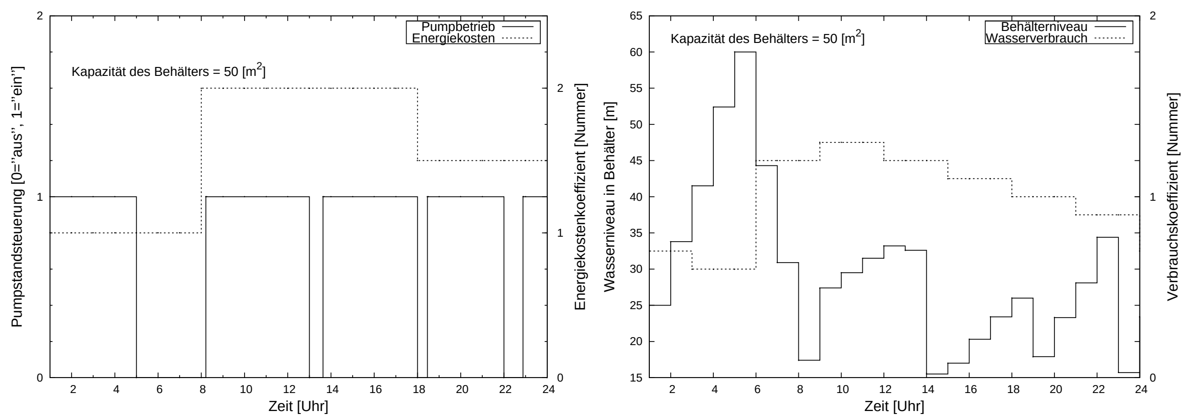


Abb. 0.7.: Fallstudie $O1$: Identifizierte optimale Pumpensteuerung und entsprechende Behälterwasserniveau für Behalterkapazität von 50 m^2

Die Simulation wurde um 0 : 00 mit einem Wasserniveau von 25 m im Behälter $N12$ begonnen. Nach dem berechneten optimalen Ablaufplan soll der Behälter in den ersten 5 h bis zur maximalen Kapazität (Wasserniveau = 60 m) gefüllt werden. Obgleich niedrige Energiekosten bis 8 : 00 dauern, können sie aufgrund der Kapazitätsbeschränkung des Behälters nicht mehr genutzt werden. Erst wenn eine erhöhte Wassernachfrage (ab 6 : 00) den Behälter teilweise erschöpft, kann die Pumpe $N11$ wieder eingeschaltet werden. Da dies den höchsten Energiekostenaufwand verursacht, wurde die Pumpe benutzt, um gerade das minimale Niveau in Wasserbehälter zu erhalten. In der folgenden Zeitperiode mit normalen Energiekosten (nach 18 : 00) wird die Pumpe auf ähnliche Weise benutzt.

Optimierung der Wasserbehälterkapazität - Wie gerade gezeigt, wird die Optimierung der Pumpensteuerung hauptsächlich durch die vorhandene Speicherkapazität des Behälters begrenzt. Deshalb ist es notwendig, die Investitionen in die Behältervolumen und die Betriebskosten der Pumpen gemeinsam zu optimieren. Die optimale Pumpensteuerung und der entsprechende Behälterwasserniveau für die berechnete optimale Behälterkapazität von 55 m^2, unter Annahme diskontinuierlicher Kapazitäten des Behälters mit 10 vorgegebenen Querschnitten und festgelegten minimalen und maximalen Wasserniveaus, sind in Abbildung 0.8 dargestellt.

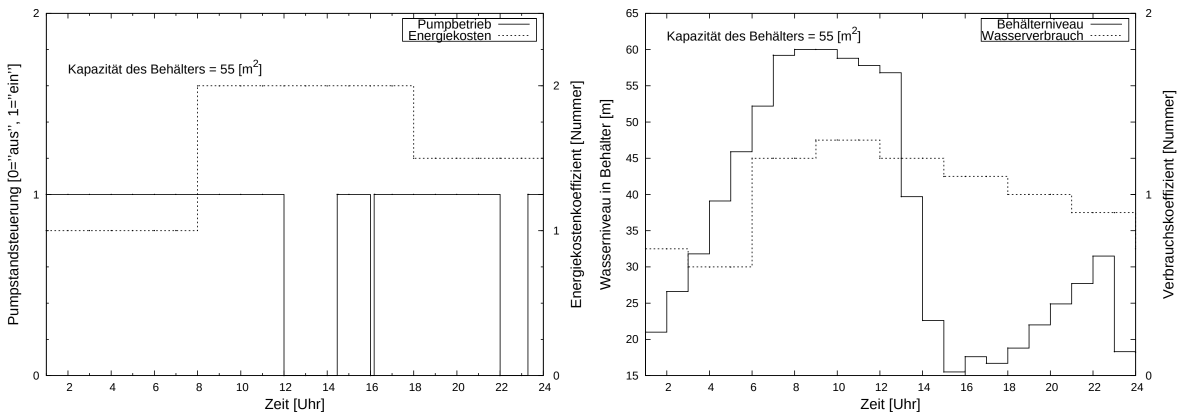


Abb. 0.8.: Fallstudie $O1$: Identifizierte optimale Pumpensteuerung und entsprechende Behälterwasserniveau für Behalterkapazität von 55 m^2

Im Vergleich zur vorherigen Lösung, zeigen Pumpenbetrieb und Behälterniveau nun viel bessere Ergebnisse. Bereits bei einer Zunahme des Behältervolumens $N12$ um 10 % (von 50 bis 55 m^2), ist eine wesentlich kostengünstigere Pumpensteuerung erreichbar. Der Betrieb der Pumpen beginnt wieder in der Phase geringer Energiekosten, bis die volle Kapazität des Behälters $N12$ erreicht ist. Nun ermöglicht die erhöhte Speicherkapazität, dass die Pumpe in der Phase der höchsten Energiekosten abgestellt wurden. Die Gesamtkapazität des Behälters ($2475m^3$) ist im Vergleich zu der Gesamtwassernachfrage ($26880m^3$) noch immer relativ klein und die Pumpe wurde in der Phase der höchsten und normalen Energiekosten auch teilweise benutzt. Dennoch sind die gesamten Investitionen in Behälterkapazität und Betrieb der Pumpen der endgültigen Lösung ($2.13*2393+0.91*262390 = 243873$ \$) um ungefähr 8 % niedriger als die der ersten Lösung (264784 \$), die nur aufgrund Pumpensteuerung optimiert wurde. Daraus lasst sich schließen, dass die kombinierte Optimierung von Pumpensteuerung und Behälterkapazitäten zu bedeutenden Kostenvorteilen führen kann. Die gemeinsame Optimierung von mehreren Parametern, die entscheidende Rolen in Betrieb von Wasserversorgungssystemen spielen, wurde hier als Haupt Leistung des Betriebsmodells erkannt.

Kurzfassung und Ausblick

In der vorliegenden Arbeit wurde eine Methodologie für die integrative Entscheidungsunterstützung in Management von Wasserversorgungssystemen entwickelt und in drei Modellen (Planungs-, Entwurfs- und Betriebsmodell) eingebaut. Das Planungsmodell integriert technische, ökologische und sozioökonomische Aspekte, die für die Auswahl der Wasserentnahmen, die Aufbereitung und den Transport zu den Wasserverbrauchern relevant sind und ermittelt eine Auswahl von möglichen Systemkonfigurationen, die für verschiedene Kombinationen von Entscheidungsträger-Präferenzen optimiert sind. Das Entwurfsmodell dient der Dimensionierung der Komponenten der Wasserversorgungssysteme, die ein im Bezug auf ökonomische Kosten und Zuverlässigkeit optimiertes Systems darstellen. Eine Ausfallanalyse und eine Analyse der Parameterunsicherheiten (z.B. prognostizierter Wasserbedarf) sind im Modell vorhanden und dienen einer risikoorientierten Abgrenzung von möglichen Entwurfsvarianten. Das Betriebsmodell identifiziert die optimale Größe von Wasserspeicheranlagen und den optimalen Betriebsplan von Pumpenanlagen, die gleichzeitig minimale Investitionskosten und Betriebskosten haben.

Alle drei Modelle basieren auf der Netzwerk-Repräsentation von Wasserversorgungsstruktur und -funktion und auf einer Kombination von *Simulated Annealing* und *Branch and Bound* Algorithmen zur Lösung des *Minimum Cost Network Flow* Problems. Fortgeschrittene *Path Restoration* und *Latin Hypercube Sampling* Methoden wurden für die Betriebssicherheit und Unsicherheitsanalyse benutzt. Alle Methoden wurden für Wasserversorgungssysteme angepasst und mit zwei existierenden theoretischen Fallstudien verglichen. Die Ergebnisse sind sehr plausibel und die angewendeten Methoden haben ein hohe Effizienz. Die Entwicklung einer einzigartigen Methodologie für die Identifizierung von optimalen Planungs-, Entwurfs- und Betriebsoptionen unter Berücksichtigung von unterschiedlichen Zielsetzungen und Kriterien, Betrachtung von Unsicherheiten sowie Integration von verschiedenen Systemparametern wird als wesentliche Forschungsbeitrag angesehen.

Eine ausführlichere Prüfung und Validierung der Modelle ist ein erste notwendiger Schritt vor der Anwendung. Die Anwendung der Modelle auf konkrete Fallstudien und die Diskussion der Ergebnisse mit Experten aus der Praxis ist erforderlich. Es muss auch erwähnt werden, dass die Vorauswahl einzelner Methoden für die Lösung des Netzproblems bezüglich der Mehrziel-Optimierung, Unsicherheit der Eingabeparameter und Zuverlässigkeit des Systems im Vergleich zu eine integrativen Betrachtung dieser Aufgaben innerhalb gut strukturierte und definierte Planungs-, Entwurfs- und Betriebsprobleme, von geringere Wert ist. Damit bleiben die hier angewendeten Methoden austauschbar, solange die Leistungsfähigkeit, die Anwendbarkeit oder die Transparenz der Methodologie gewährleistet ist.

1. Introduction

The following chapter introduces the motivation for this study, states the problems that are aimed at and defines the research objectives. It concludes with a short description of the structure of the study.

1.1. Motivation

Throughout the centuries of our society development water supply systems have been increasingly built with the aim to satisfy the ever increasing needs for clean and easily accessible water supplies. The expansion of industrial production, irrigated agricultural areas and human population in the 20th century, caused a rapid increase in water demands (Figure 1.1), that resulted in the development of ever more and more ambitious systems for capture, treatment, transport and distribution of natural water resources. Today, water supply systems are often composed of numerous ground, surface or spring water intakes, have very complex transport and distribution networks and include very sophisticated treatment facilities. The effective management such complex and cumbersome man-made systems has become a very challenging task hardly achievable without the assistance of modelling tools. Although many models for the simulation and optimisation of these systems already exist, there are only few that aim at **integrated decision support for all management stages**.

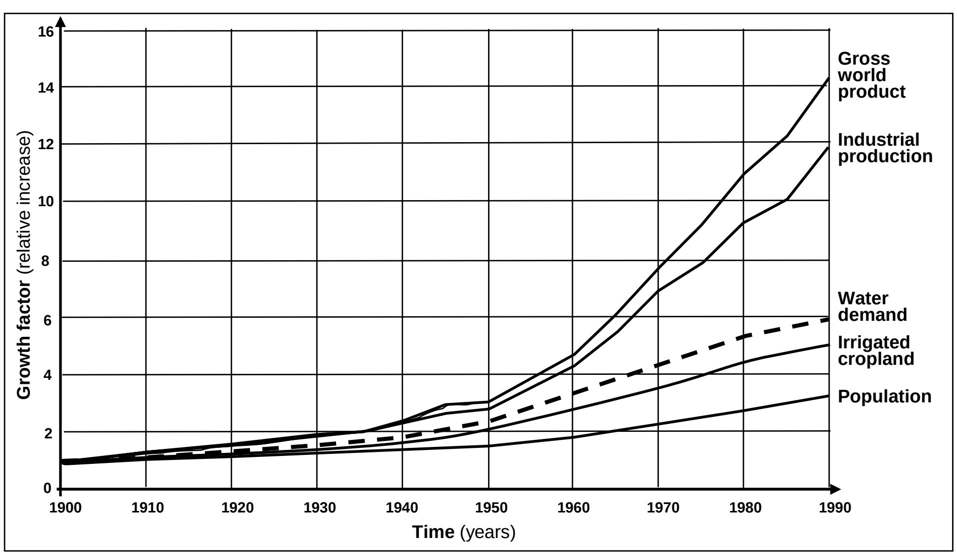


Figure 1.1.: Relative growth of world population, gross world product, industrial sector, irrigated area and water demand [source: Hoekstra, 1998]

The expanded use of natural water resources and the world wide pollution of this precious asset left behind many contaminated natural water bodies and destroyed ecological habitats. ”The growing tension between intensive water use and the functioning of natural ecosystems has shifted our perception of water supply systems from *human utility services* toward *coupled human-natural systems*” (Allenby, 2004). The integrative consideration of the natural environment and the human built-in systems has become our society’s new paradigm (IUCN et al., 1980; UN, 1992). Since infrastructural systems provide the flow of resources from the environment to the society and its economy and return not any more useful matter again to the environment, they can be seen as the meeting point of society development goals, its economic prosper and environmental protection needs (Figure 1.2). But balancing among social, economic and environmental goals is a very demanding task, not only due to the complex structure of decision making, but also due to very different temporal, spatial and value units and scales of different processes of influence that take place in these three domains. Nevertheless, the need for **integrative analyse of technical, economic, environmental and social aspects of infrastructure systems, in particular water supply systems**, represents the main motivation for this study.

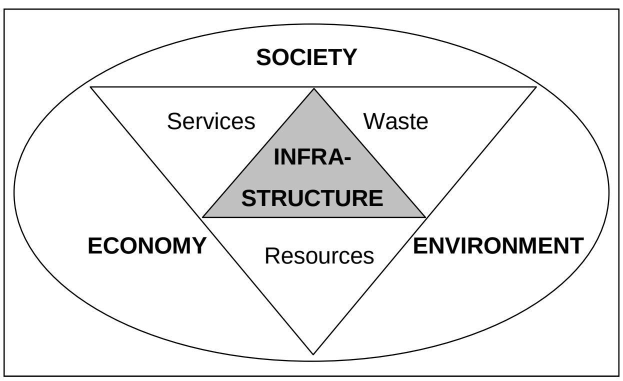


Figure 1.2.: Integrative approach to the analysis of infrastructural systems [adopted from UN, 1992]

Modern management of water supply systems implies not only the use of best practice technical measures, but also requires the application of advanced operation research methods and computer tools for analysis, evaluation, forecasting, control and optimisation of the systems. In order to identify sustainable management decisions for these complex systems, it is necessary to have tools that can create and examine different possible alternative plans and select the ones that are optimal according to predefined management objectives and preferences of decision makers. The whole process of the identification of management objectives, decision variables and criteria, through data collection and processing, to the creation and identification of ”optimal” management options is often referred as decision support (Figure 1.3). The necessity for methods and tools that enable **multi-objective approach to the management problems and integrate preferences and risk perception of of decision makers** in the development of optimal alternatives is a particular area of interests in this study.

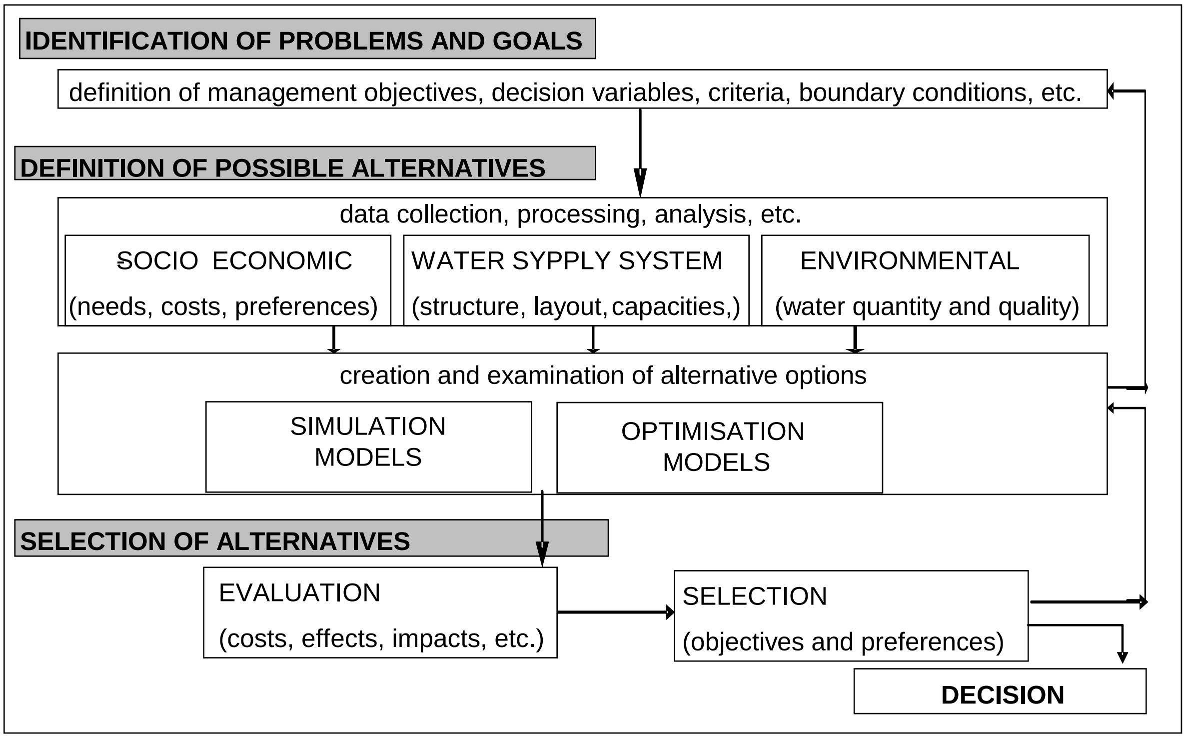


Figure 1.3.: Decision support in management of water supply system [adopted from Loucks and da Costa, 1991]

Since different management stages (i.e. planning, design, operation) have different objectives and deal with different problems, an attempt will be made to develop a methodology general enough to be applicable in different stages but still to allow ease accommodation for specific management objectives and problems. Therefore, **three computer models, namely planning, design and operation** model, will be developed based on the same methods but accommodated for each management stage. They should illustrate similarities and differences of decision support in different management stages. Furthermore, they should simplify the use of the methodology and promote its applicability. It is hoped that the use of methods and models that help decision makers to find optimal trade-off among different objectives and allow transparent dealing with costs, impacts, risk and uncertainty, within the evaluation of existing systems and the development of new ones, will contribute to the development of more sustainable water supply systems and will bring them one step further toward *integrated human-natural systems*.

1.2. General Objectives and Current Problems of Interests

Following the ideas of sustainable development (IUCN et al., 1980; UN, 1992), the analysis of water supply systems has to take into account all effects of intended activities on the environmental and socioeconomic processes of importance. In addition the money and energy flows as well as the social preferences that often govern these processes have to be considered at the same time. The development of integrative methodologies for the joint analysis of technical, environmental, economic and social aspects of water supply systems is the first prerequisite for this. Therefore, the **integration of different objectives and criteria in the creation of alternative water supply management options** is the prime problem to be dealt with.

The importance of the stakeholders participation in the decision making process has been recognized and already institutionally implemented in most of developed and many of developing countries (UNEC, 1998). For the management of water supply systems this means not just better information of public and regulatory authorities about provided water services, but also the participation of public, government, industry, environmentalists, and other stakeholders. This increases not only the complexity of the decision making process but also the importance of **the formulation of alternative solutions that encompass interests and objectives of different stakeholders and decision makers**. The implementation of the multi-criteria evaluation techniques in the analyses of water supply systems represents the next milestone of this study.

The real life driving forces, such as different water needs, variable natural distribution of water resources, various social and political preferences and different economic and technical capabilities, led to the development of many different types of water supply systems. Although these systems may differ in technical specifications, natural conveniences, form of ownership or type of management body, under the current paradigm of the Integrated Water Management (UNESCO, 1987) and the ever increasing standards for water quality and control, even the smallest water supply systems can be hardly any more considered in isolation. In addition, in the last decades, there is an obvious trend of mutual interconnecting among water supply systems, due to the factors such as saving from the economy of scale, increasing reliability of water supply, easier transfer of know-how and simpler regulatory control (Hirner, 2001 presents the performance assessment and Rott, 2005 and Rott, 2006 the current trends in the water supply sector in Germany). Although many sophisticated modelling tools for the analysis and management of such ever larger and complexer systems have already been developed, very few have been practically implemented (Goulter, 1992; Walski, 1995). Accordingly, the problem with the analysis of water supply systems is not the lack of appropriate tools, but rather a challenge to select methodologies that are able to **handle often very complex problems with simple enough and easily understandable methods** (Walski, 2001). The identification of such methodologies with the aim to increase the understanding and applicability of the System Analysis techniques in the management of water supply systems is intended to be the main practical contribution of this study.

In addition to the ever increasing spatial dimension of water supply systems, their inflexibility poses even greater problem to their operators and managers. Water supply systems are typically designed for periods of 30 to 50 years and very often function much longer. Due to the natural variability of most of their input parameters, their uncertain character and constant changes in their environment, it happens quite often that water supply systems work under different conditions than planned for majority of their life-time. For example many water supply systems in developed countries operate in a low efficiency range due to the reduced water consumption in last years (Tillman et al., 1999). In contrast, in developing countries, majority of water suppliers still struggle to keep the peace with the rapidly increasing water consumption. The physical changes in the systems characteristics due to corrosion, deposition, hydraulic stress, etc., additionally contribute to the variable and uncertain environment in which the systems operate. Therefore a huge interest in the development of methodologies for a more **robust, flexible and reliable water supply systems planning, design and operation** with alternative options that are better accommodated for different possible development scenarios exists. In particular, the incorporation of reliability in the water supply system design is an important issue that will be addressed in this study.

There are many socioeconomic processes that influence the recent changes in the water supply sector. Liberalization and globalization of the water market, privatization of public water companies, tighter environmental and water quality standards and greater environmental awareness are just some of the pressures that dictate systems efficiency increase, cost saving, environmental impacts attenuation and better accommodation to the users needs. Although, water consumers are still accustomed to the very comprehensive services, and are still willing to pay for them, it is reasonable to expect that their preferences, priorities and expectations may also change in the near future. Since the traditional design approach, based on the use of standards and codes of practice, is not able to account for variable system performance evaluation, the alternative approaches, such as Stochastic Design, have already been suggested. Furthermore, novel approaches provide for the much more transparent and precise quantification of the system uncertainty. **The incorporation of the uncertainty considerations, users' and decision makers' expectations and risk tolerance into the development of alternative water supply systems planning, design and operation options** is a next important problem that this study aim at.

Finally, water supply is a very specific industry that is at the same time driven by the "equity principle" and "economic efficiency". Water is a basic human need and the provision of drinking water is mainly defined as a constitutional obligation of a state. In contrast, the economic efficiency of water supply systems is an important factor that determine their future development. Due to the fact that water users are connected to only one water supplier there is no free water supply market that can regulate the water price by the principle of offer and demand, the water suppliers are often strictly controlled by the authorities to meet user demands at non-profit or low profit prices. In such a set-up, it is of prime importance to provide for a sustainable decision making and active participation of all involved stakeholders. Therefore the assessment of initial investments or production, maintenance and system expansion costs, as well as the assessment of the benefits of the water provision, have to be very transparent, apparent and evident. Consequently, each step of the suggested methodology has to be **transparent and easily applicable to real-life water supply systems**.

1.3. Specific Objectives and the Aim of the Research

According to the general problems of interests stated above, a specific research objective of this study is **the development of a modelling methodology for the multi-objective and risk-based decision support in planning, design and operation of water supply systems**. The methodology should be systematic, integrative, transparent, and applicable to already existing water supply systems and new ones. In essence, the methodology should be able to address the following issues:

1. Efficient modelling representation of the system characteristics,

2. Integration of multiple objectives and multiple preferences,

3. Integration of uncertainty, risk and reliability considerations.

As for any other study that aims to develop a methodology for the analysis of a real system by replacing it with a **model** [1] the first obvious objective is to find an **appropriate conceptual and computational representation that will well enough substitute not only the characteristics of water supply systems but also incorporate the objectives and specifics of the management problems**. As far as the characteristics of water supply systems are concerned, beside their structural components such as intakes, treatment plants, storage and delivery facilities with their characteristics such as locations, flow capacities, pressure conditions, etc., the model has to represent the main processes such as water withdrawal, transport, treatment, etc. and the different modes of operation such as design conditions, normal operation and failure modes. Furthermore, a life time of water supply systems consists of different management stages, such as planning, design, operation, expansion, rehabilitation, that set up different objectives for the analysis. The methodology should not only be flexible enough to accommodate for these different stages but it should also encompass a wide range of combination of preferences toward different objectives that can be set by different stakeholders and decision makers. Finally, in order to increase the applicability of the suggested modelling methodology, it has to stay simple enough and easily understandable.

Secondly the suggested methodology should provide for the **integration of various objectives and criteria into analysis of water supply systems**. Since there are a large number of environmental and socioeconomic impacts and factors of influence, the most important ones have to be identified and their functional dependencies in terms of losses (costs) and benefits (contributions) to and from water supply systems have to be quantified. Furthermore, in order to integrate the functional relationships of different types of costs and benefits (e.g. environmental losses due to water extraction, social benefits due to water provision, etc.) into one computational model, they have to be brought to same associate units and scale. Since different decision makers have different preferences toward environmental, economic or social criteria, **the possibility to encompass such varying priorities has to be provided**. Furthermore, the methodology should provide for the trade-off among

[1] a theoretical construct that represents real life structures and processes with a set of variables and a set of logical and quantitative relationships between them

different objectives with the aim to promote the identification of the alternatives that are acceptable for all decision makers.

Lastly, **the integration of uncertainty, risk and reliability considerations into evaluation of water supply systems performances** is of a prime importance for the practitioners. Unfortunately, knowing that most of the uncertainties such as data's, system's, model's uncertainties are inevitably connected with modelling, it is not to expect that the "true" system performance can be assessed in advance. Still, the mediation of accessible uncertainties represents the basis for robust and flexible system design. The traditional design approach quantifies uncertainties based on standards and codes of practices and accounts for them by adding some spare capacity in order to "be on the safe side". An intention of this study is to offer a methodology that will evaluate the system performance for the recognized level of input data uncertainty and quantify its reliability accordingly. In addition, the users' and decision makers' preferences and risk tolerance will be implemented in evaluation of system's performance in order to define "how safe the system need to be". Based on decision makers' risk acceptability, the statistical evaluation of system performances for large number of system simulations with uncertain parameters will yield reliability of a system. Such calculated reliability may be then traded-off with other criteria such as economic costs or environmental impacts.

1.4. Course of Action

The main building blocks of the presented study are arranged in three following chapters.

Chapter 2 - The Theoretical Foundations - defines the notation, introduces the main characteristics of water supply systems and defines the main concepts necessary to achieve the objectives of the study. Physical, socioeconomic and environmental characteristics as well as uncertainty, risk and reliability issues of a prime importance are presented and discussed. The hierarchical approach to the management of water supply systems is presented and the suggested division into planning, design and operation stage is adopted. For each of these stages a detailed literature review of the application of the System Analysis techniques is provided and discussed with the aim to identify the starting point and the needed focus for this study.

Chapter 3 - The Methodology Development - chapter aims to establish the theoretical base for the development of a methodology that will enable integration of environmental, economic and social aspects in water supply development as well as the uncertainty, risk and reliability based assessment of water supply systems performances. In order to provide for a good structural and functional representation of the systems, the methodology is based on the Network Concept and combines the Graph Theory algorithms with the advanced System Analysis optimisation methods to achieve the effective solution of the water supply management problems. Suggested algorithms are accommodated to deal with water supply planning, design and operation problem and implemented into corresponding models. Special attention has been devoted to the multi-objectiveness, transparency and applicability of the methodology.

Chapter 4 - The Model Development and Application - chapter presents three computer models based on the previously defined methodology. The planning model provides for the integration of technical, economic, environmental and social objectives in the process of development and selection of new water supply strategies (e.g. possible new sources, allocation to demand centres, water transfer options, etc.). The participation of stakeholders is assumed and the identification of the optimal systems configurations for different sets of stakeholders preferences is aimed at. The design model deal with the minimum cost sizing of water supply components and the system's reliability issue. The deterministic design criteria are combined with stochastic evaluation of the parameters' uncertainty in order to obtain the systems alternative options whose performance satisfy some predefined failure scenarios (e.g. component failure scenarios, fire fighting, etc.) and provide for some predefined uncertainty level of input parameters (e.g. demands, hydraulic performance, etc.). Based on the risk acceptability of the decision makers the final design option may be selected as a trade-off among system reliability and its costs. Finally, the operation model is intended for the extended-time analysis and optimisation of the storage capacities and pumping schedules of water supply systems. Each model is applied on two theoretical case studies. The first serves to demonstrate model capabilities and the second to validate and compare its efficiency with already existing models.

2. Foundations of the Study

This chapter explores the main physical, socioeconomics and environmental characteristics of water supply systems that are relevant or related to the study objectives, provides definition of the basic terms that will be further used and presents the main issues of importance that will be addressed. Due attention is devoted to the identification and quantification of the most important environmental impacts and the prime socioeconomic aspects that are to be considered. Furthermore, the approach to tackle the uncertainty, risk and reliability issues is provided. Finally, the management of water supply systems is broken down to the planning, design and operation stage, whose analyses are understood as the optimisation procedures. At the end of the chapter, the current state-of-the-art in planning, design and operation analysis of water supply systems is provided.

2.1. Main Characteristics of Water Supply Systems

Although water supply systems range from an individual well and stream intake, used since early times, to the large comprehensive multi purpose systems for water production, purification and distribution, their general role can be defined as **spatial and temporal reallocation of water resources from nature to human society, keeping in mind quantitative and qualitative aspects of water availability and human needs**. Such definition already reflects not only the importance of the provision of clean water for the general prosperity of our society but also the significance of the environmental and water availability concerns. After a short introduction into the main physical characteristics of water supply systems, a review of environmental and socioeconomic issues of importance follows.

2.1.1. Physical Characteristics

Water supply systems are usually classified into an acquisition, a treatment and a delivery **parts**, or **components**, that are composed out of the following main building **blocks**, or **elements**: source, raw water storage, treatment, storage, distribution and use or delivery area (Figure 2.1). Although in many instances, some of these components are not necessary (e.g. for systems with groundwater sources the raw water storage or even the treatment facility can be often omitted), large water supply systems are usually a very complex conglomeration of many such components and consist of more than one sources, treatments units, pump and storage facilities. Transport of water among these components is provided by transmission (trunk) mains and connected appurtenances (i.e. pumps, valves, fixtures, etc.). The distinction between **the transmission system** that transport water between components of the system and **the distribution system** that distribute water in a supply area and deliver it to the end user is very important to mention, since only transmission system will be considered in this study.

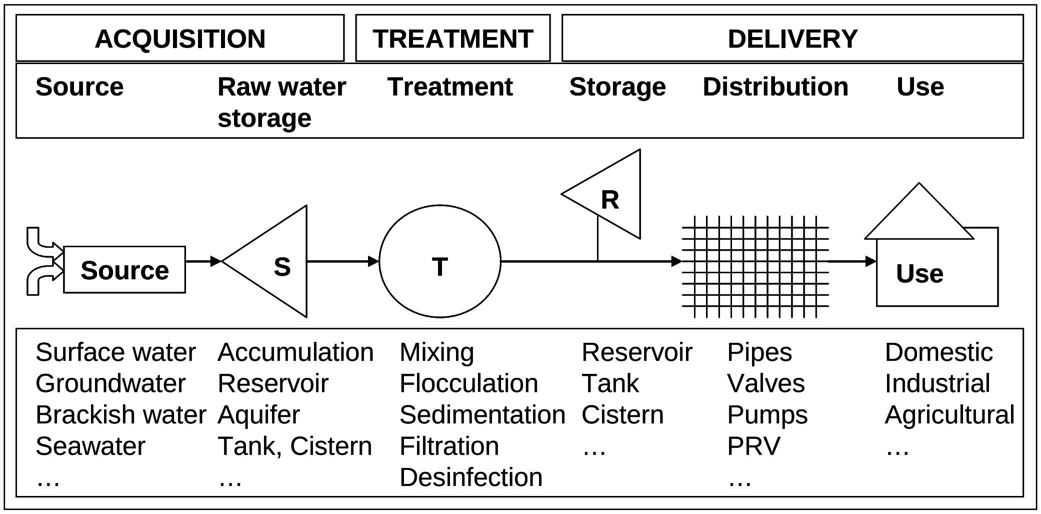


Figure 2.1.: Components of water supply systems [source: Grigg, 1986]

The way how the system elements are connected, so called **layout**, forms next important physical characteristic. Transmission and distribution systems can be either branched, semi-looped or looped (Figure 2.2). In semi-looped and looped systems there may be several different paths that transmit water between two components, while in branched systems there is only one. Although branched systems are much more economical, the looped ones provide additional redundancy[1] and are preferred, not just for the distribution networks layout but for the transmission system layout as well. Although theoretically a very large number of possible system layouts may exist, practically the number of potential links among components is constrained by terrain configuration, physical feasibility of a link construction, cost of additional links and the needed system reliability level.

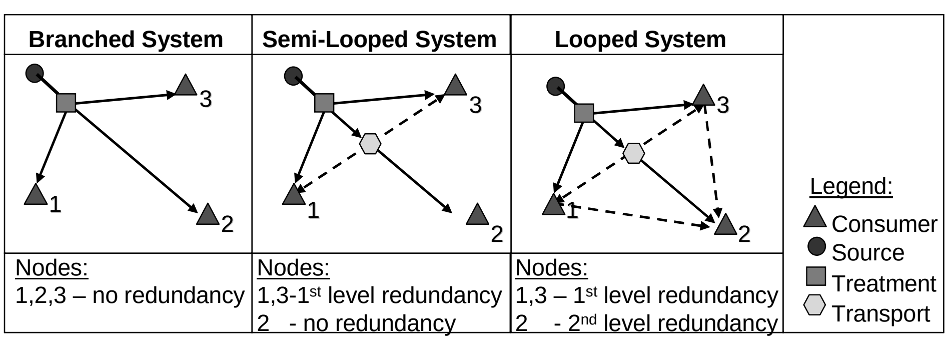


Figure 2.2.: Branched, semi-looped and looped layout

[1] 1^{st} - level redundancy means the existence of one additional path able to supply a node effected by a failure of some link, 2^{nd} - level redundancy means the existence of two paths and so forth

In addition to the existence of the paths between sources and consumers, in order to transport demanded quantities of water they have to have enough **capacity**. The flow of water within a system is determined not only by the layout and capacities but also by the energy input, energy losses and the state of control elements such as valves, overflows, and others. Water flow is an unique system parameter that is dependent on almost all other physical properties of a water supply system.

Finally, all elements of a system with their capacities and arrangement in a layout will be collectively referred here as **configuration**. Althogh many different configuration may provide very similar system performance, they often differs a lot in economic, social or environmental aspects. Therefore, it is necessary to consider all these aspects by the development and selection of system configurations. In addition, the physical characteristics have to be considered as variable and uncertain parameters since many of them change during system's life time, for example changes in friction coefficients due to deposition and corrosion, leakage and losses in transport and distribution, changes in pumps and valves characteristics, etc.. These changes have to be considered already during the development of new system configurations.

2.1.2. Water Supply

Water sources, such as springs, rivers, lakes and groundwater aquifers, represent the beginning points of water supply systems at which raw water enters. The water availability at the sources significantly influences the characteristics and operation of water supply systems. Storage and transmission facilities are used to compensate for the different spatial and temporal distribution of natural water resources and human demands, while treatment facilities purify water to the level of the drinking water quality standard. The natural variations such as oscillations of groundwater level, changes in river water quality, extreme events such as droughts and floods and ever increasing anthropogenic influences such as the pollution of water resources make availability assessment of water supplies very complex and uncertain.

2.1.3. Water Demand

At the other end of water supply systems are the consumers. The way how they use water is the main driving mechanism behind the systems function. Water demand vary in time (hourly, daily, monthly, seasonally and yearly). In addition it can vary in space as the consequence of population increase, decrease or migration, or different development trends or changes in industrial and agricultural production. In effect, water demand is dependent on technical (e.g. pressure distribution in a water supply system), natural (e.g. climate and weather conditions), social (e.g. institutional arrangement of water provision, habits and customs of water users) and economic (e.g. water price and economic status of the users) characteristics of a supplied region. Being influenced by so many factors it is not surprising that the water demand is the most variable and uncertain parameter in the water supply systems' analysis.

2.1.4. System Performance Measures

Present water supply systems range form small scale systems with a single source, no treatment and simple transport system to large regional systems which comprise numerous ground and surface sources, treat water at complex treatment facilities and deliver it to large distribution networks that consists of many reservoirs and pump stations, thousands of pipes and pipe fittings, various controllers and measurement devices and have very sophisticated operation and management systems. Although numerous performance measures are of importance for the functioning of such complex systems, from the technical point of view, the two most determining ones are water flows and pressures. They represent the essence of any quantitative analysis of water supply systems and will be adopted as the main indicators of a system's performance.

2.2. Environmental and Socioeconomic Issues of Importance

Until the last decade, aside from rare examples such as a three volume series on the social and environmental effects of large dams of Goldsmith and Hildyard (1984, 1986, 1992) and the revision of large water project impacts on low-income rural communities in subtropical and tropical river basins of Biswas (1996), there were not many attempts to quantify impacts of water supply systems on the society (for overview see Scudder, 1996; Chadwick, 2002). Such a need emerged only when the degraded state of the environmental quality threaten to either directly or indirectly endanger the human's health and the future prosper of our society. Today there is an increasingly large number of literature about negative environmental impacts of large scale water projects. Some large scale examples of the depletion of natural water sources, such as the Ogallala aquifer in the USA (Wilhite, 1988), the Yellow river in China (Zhu et al., 2004) or the Caspian, Aral and Dead seas (Kobori and Glantz, 1998) in Eurasian region, and thousands other smaller examples of the pollution of water ecosystems, led to the development of large number of methods for the environmental impacts assessment (EIA). An overview of these methods can be found in Yurdusev (2002). Essentially each method consists of two basic steps: a) identification of impacts and b) their quantification. The same two steps are adopted for the integration of environmental and socioeconomic issues in the water supply systems' analysis.

2.2.1. Environmental Impacts of Water Supply Systems

It is broadly adopted that engineering projects may have impacts on the full range of environmental components, including air, water, land, ecology and noise as well as on the physical processes that occur in the environment (CIRIA, 1994). Experience suggests that the effects of large scale projects have to be considered on three time scales: a) during construction, b) upon completion and c) over the period of exploitation, and on several space scales: a) immediate surroundings, b) the neighbourhood, and c) wider possibly affected areas (Munn, 2006). In addition, impacts may be directly attributable to the project (e.g. lowering of the groundwater table due to water withdrawal) or indirectly caused (e.g. land degradation due to the building material excavation). Although environmental impacts of an engineering project are very site and project specific, the study of Construction Industry Research and Information Association form England (CIRIA, 1994) provide a good overview of the possible environmental impacts of water supply systems on air, water, land and ecology (see appendix A.1). Based on this study, the most common environmental impacts of water supply projects are summarized in the following.

Air quality in the neighbourhood of a water supply system can be temporarily affected during the construction by entrainment of dust from aggregate stockpiles and haulage roads or permanently affected by changing the micro-climate around accumulations such as raw water storages. Furthermore, such open water surfaces increase evapotranspiration rates that may influence the vegetation in the area or increase the frequency of fog and mists. The potential impacts are usually assessed by calculating the water balance with and without accumulation.

Abstractions from groundwater aquifers and rivers reduce the **water** amount available in these systems and in extreme cases may lead to the depletion of aquifers, the loss of river base flow and the devastation of wetlands and other ecosystems. Furthermore, reduced water quantity in natural systems influence its quality and promote development of higher concentration of pollutants and nutrients. In addition, river impoundments and water supply accumulations may influence not just river flow regime but may also rise the groundwater table and influence the interactions among surface and groundwater bodies. Application of standard hydrological and hydrogeological methods for the balancing of the water resources is the most often used way to determine the allowable water withdrawal quantities.

Building of accumulations and objects as well as installation of pipelines, cause the loss of **land** resources and may impact ecological sites and the open space amenity value. Large water impoundments and accumulations may, in addition, cause slope failures or increase pressure in geological fault zones. In contrast, water supply intake places are usually protected by zones of reduced human activity in which the natural state of the resources is protected.

Water accumulations cause not just permanent loss of flooded **habitats** but significantly impact upstream and downstream geomorphological processes and habitat conditions. Decrease or increase of river velocity may favour some species at the expense of others and physical barriers and loss of high flows may cease the migration of some fish species. Reduction in available groundwater and river water amounts and reduction of their natural variations, may lead to changes in the ecology of river corridors, estuaries and wetlands. Habitat and ecological studies may to some extent assess these changes.

2.2.2. Quantification of Environmental Costs and Benefits

In order to avoid the above stated negative environmental impacts it is necessary to assess the state of an ecosystems prior to a project and assess the possible changes that may be caused by a project. Furthermore, in order to be able to make a comparison among project alternatives and to make trade-off with other project objectives a quantitative or qualitative categorization of the value of the environment has to be made. Since this is not an easy task, the environmental impacts of a project or an action are often expressed through indicators [2]. A very complete list of environmental indicators of importance in relation to water resources projects has been published by UNESCO (1987). But the quantification of the impacts of a project even using indicators is still a very complex problem. While some environmental indicators, for example the decrease in a groundwater level, are quantifiable or measurable, others such as recreational or aesthetic value of the environment, can be only qualitatively expressed. In order to overcome such differences some EIA methods use qualitative evaluation for all indicators (i.e. ad hoc, checklist or overlay methods), some avoid expressive quantification by establishing direct dependencies among project activities and environmental indicators (i.e. Matrices and Network technologies) while some transform all impacts to monetary terms (i.e. Benefit-cost analysis). The first group is often judged as too rough and vague since it a) evaluates indicators mainly by auditing experts, decision makers and other parties of interests that express their subjective opinion and b) because it uses qualitative values, such as "good" and "bad" that may have different meanings for different participating parties. Placing a monetary value on environmental impacts is based on the assumption that individuals are willing to pay for environmental gains or, conversely, are willing to accept compensation for some environmental losses. Such techniques are not just subjective to the individual preferences (Pearce and Markandya, 1993) but one has to keep in mind that the willingness to pay or the willingness to accept should reflect the preferences of future generations and other species and are extremely difficult to forecast (Beder, 2002). In addition, the market value might not be consistent with long-term welfare or survival of society, since the economy is interested in the environment only to the extent that it can ensure a continuous supply of goods and services to meet human wants (Beder, 2002). Finally, the approach based on the establishment of the direct dependencies among project activities and environmental indicators is selected as the most appropriate.

The assessment of changes of some environmental indicator as a consequence of some projects or actions is a very complex task that often requests complex studies. Stated that the enclosure of a broad range of functional relationships between water supply project properties (e.g. withdrawal rates, transported quantities, etc.) and their environmental impacts (e.g. groundwater level, river water flow, etc.) within one systematic framework is the prime focus of this study, instead of the development of the models for the assessment of individual impact it is assumed that the dependencies of the indicators states from the project properties or some project actions can be represented as simple single-variable functional relationships. Since these relationships are to be used for the selection among alternative project parameters or actions, they do not need to represent the environmental impacts in absolute values but can only represent the relative difference among different project parameter values or actions. Similar as the other engineering parameters, such as expected water demands or estimated operation costs, the environmental impacts may be approximated by: a) statistical evaluation of existing data, if available, b) transferring of the results from similar studies, c) using existing knowledge about natural processes (e.g. impacts of the lowering of a groundwater level on the surface vegetation or decreased river flows on fish species) or d) different kinds of trends analysis and logical deduction. In addition it is argued that a simplified functional relationships between environmental impacts and project parameters may have the accuracy

[2] numbers or ratios that help to reveal the status and changes of selected parameters

of the similar order of magnitude as most other input parameters (e.g. prediction of vegetation cover reduction upon decrease in groundwater level has a similar order of accuracy as the prediction of economic costs of installation and operation of pump station in some future time period) and that the approximate but systematic evaluation of a broad range of environmental effects may be much more beneficial than a more precise analysis that is focused on few impacts only. Furthermore, the accuracy of the EIA is dependent on the stage in water supply management it serves for. While for the planning phase the general trends and tendencies may be enough to identify the most sensitive environmental areas that is to be preserved, the design and operation phase will need much more accurate functional relationships among environmental indicators and parameters of a system. But in these phases much more data, time and resources may be available for the EIA and the functional dependencies can be much better accommodated to a particular site or even detailed simulation models may be developed.

2.2.3. Socioeconomic Aspects of Water Supply Systems

The **social** impacts analysis can be defined as an analysis of project impacts on sociocultural systems (SCOPE, 1972). Beside obvious benefits, such as improved hygienic-health conditions and better living standards, water supply projects may cause non-desirable migrations of population toward places where the systems have been built, loss of populations primary activities or the changes in population habits and customs. In addition, under the circumstance of good water availability a trend for the not-beneficial water use and its dissipation often develops (e.g. cars and street washing). Although some general dependencies between water provision and its social impacts may be reasonably assumed (e.g. provision of water attracts new population) the assessment of the more detailed social impacts is almost an impossible task. Furthermore, for most types of the engineering analysis much more important social aspects are the preferences of the investors, public or authorities that are not the consequence of a project or actions but are input parameters to the analysis. These preferences are often the determining factor in the "choice" between project alternatives and have to be considered within the integrated decision support.

Water supply systems provide support for many important economic activities such as the agriculture, livestock and many other industries. Furthermore, the **economic** aspects, such as the economic benefits of water provision or the investment and operation costs of the systems elements are still the main decision criteria in planning, designing and operation of water supply systems. In contrary to other aspects, economic costs and benefits can be assessed for each alternative systems configuration or management option in monetary units. Even more, the economic analysis methods, such as Present-worth, Rate-of-Return, Annual-cost and Benefit-Cost Ratio methods (James and Lee, 1971), allow for the scaling of different costs in time scale and enable their mutual comparison.

A state's **institutional** organization, its form of government, laws and customs constitute the framework within which society functions and directly effects the water resources management. Although different forms of water supply companies may provide water within a country, in most cases water supply undertakings come under extensive governmental control exercised through legislation, regulations, standards and inspection procedures. These effect the objectives, methodological approaches, financial capabilities and operation standards of water supply providers but are extremely hard to assess and quantify and will not be further considered in this study. .

The **financing** of water supply infrastructure, especially of large scale projects, was traditionally a task carried out by the public sector through forms such as direct investments, subsidizing, crediting, and others. Although in most of the world countries the provision of water is still a public responsibility, in the last decades, there is an increasing involvement of the private sector through various forms of management agreements, lease agreements, concession or full or partial privatization. The form of ownership and financing may largely influence the selection of criteria and decision alternatives in management of water supply but are also extremely hard to quantify and will not be further considered in this study. Nevertheless the inclusion of the preferences of decision makers in the alternative selection process allows to incorporate to some extent the institutional and financing aspects.

2.2.4. Quantification of Socioeconomic Costs and Benefits

The way of managing, investing in and thinking about water resource projects is a consequence of complex social processes such political conditions, social preferences, trends in science and many others that are constantly taking place in the society. In addition, technological improvements such as the development of water saving appliances, changes in social norms such as the increased environmental awareness and global changes such as climate change, may significantly influence not only the water demand but also the social benefits of water use. Some of these primary effects can be directly connected to the parameters of the water supply systems while most of the secondary effects such as the provision of new jobs, resettlement of population, migration to the urban areas are to case specific to be generalized in the functional relationships.

Economic benefits of water supply systems are usually assessed based on the economic value of used water, often referred as *willingness to pay*, and are either calculated directly analysing the economic process or by covering from the loss functions of water shortages (e.g. the value of water used in the food industry may be much greater that the one of water used for cooling purposes since in the second case water may be easily replaced with some other liquid or by using recirculating techniques). As far as economic costs are concerned two main types of costs are of a prime concern: capital (fixed) and operation (variable) costs. For each potential water supply systems component, these cost can be calculated from the characteristic of the components (e.g. dimensions, capacities, etc.), conditions of installation or operation (e.g. terrain, climate, etc.), prices of material, machinery and labour, and economic and financial conditions, such as availability of credits, rates of interests, and others. Since this is a very cumbersome process, costs are often approximated with cost coefficients (fixed costs per unit dimension of a component) or cost functions (functional dependency of costs and size of a component) that are obtained either by statistically analysing costs of already built systems or analysing current prices at the market.

2.3. Uncertainty, Risk and Reliability in Water Supply Systems

In the analysis of water supply systems the term **reliability** typically implies measuring of the ability of a system to meet consumer requirements in terms of quantity and quality under both normal and abnormal operating conditions (Mays, 1996a; Ostfeld and Shamir, 1996). Thus, reliability is conceptually related to the probability of a system not-failure. Xu and Goulter (1999) identify three main types of failures: 1)Component failure, 2) Demand/Supply variation failure and 3) Hydraulic failure. The rate, occurrence and consequence of a failure can be measured in several different but related ways, depending on the needs and relevance of the particular aims of an analysis (Goulter, 1995; Mays, 1996a). Although reliability has been for a long time recognized as one of the prime issues in the water supply sector (Goulter, 1987; Walters, 1988), Shamir (2002) still identifies two imperative problems connected with it: "the non existence of standardized and widely accepted criteria for defining and quantifying reliability and the non applicability of the existing methodologies for incorporating reliability measures and criteria into procedures and formal models for management of water supply systems". Even more, the same author suggests that the reliability criteria should be defined "from a point of view of the consumer, and should reflect the cost of less-then-perfect reliability".

Shamir (2002) schematically presents the "cost of less-then-perfect reliability" as on left graph in Figure 2.3. He identifies the large cost increases necessary for the improvement in initial reliability and for the achievement of the extremely reliable systems. In between these two extremes, he depicts the flatter portion of the curve, where the proportionally large increase of reliability may be achieved with modest cost increase. Accordingly, it is reasonable to expect that the range of interest in terms of cost of less-then-perfect reliability lies at the end of the flat part before the curve sharply bends upwards.

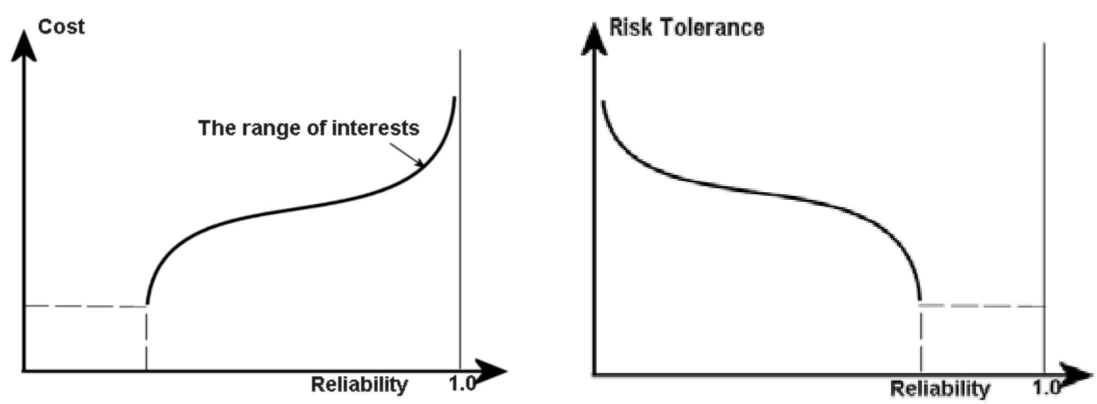


Figure 2.3.: Shematised cost-reliability [source: Shamir, 2002] and risk-reliability curves

But the system reliability, or the needed level of system reliability, is also a subjective category that may differ for different decision makers (e.g. water users, water companies, politicians, etc.). Their attitude toward system reliability is in general defined by their risk tolerance or "acceptability of less-than-perfect reliability". This willingness to accept the probability of a failure can be schematised as on right graph in Figure 2.3. A very high risk tolerance would practically mean that the user is ready to accept systems with very low reliability while very low risk tolerance demands for extremely reliable systems. In between these extremes, it is logical to expect that a medium risk tolerance range exists in which some substantial improvements in reliability may be achieved for small sacrifices in risk tolerance (e.g. reductions in hard constraints such as minimum pressures often lead to significant cost savings in water supply system design).

Important to conclude from the previous considerations is that the reliability of a water supply system can be traded with the system costs only if decision maker's risk acceptability is considered. In most of the traditional engineering the acceptable risk levels are set up by standards and codes of practice that are devised to provide good functioning of a system plus some safety margins. According to the previous, if this standards are to high or if acceptable risk level is too low, the optimum "cost-reliability" range form left graph in Figure 2.3 may not be considered at all. A comprehensive design theory that replaces deterministically defined design criteria with the probabilistic one and enable incorporation of risk perception into design analysis is called Stochastic Design (Henley and Kumamoto, 1981; Ang and Tang, 1984; Plate, 2000) and will be used in this study.

Nevertheless, if looked at the main modes of water supply system failures, it seems that the Traditional Design is very practical for the first mode (component failure), while the Stochastic Design seems to be much applicable to the second and third mode (demand/supply variation and hydraulic failure). Since the last two basically represent the failure of the system performance due to variation or uncertainty in demands, supplies or hydraulic parameters, following two analysis will be done:

- component failure (physical failure of some individual system component),
- performance failure (failure in system performance due to variability or uncertainty).

For the component failure analysis [3] the traditional design approach is very convenient since these extreme conditions can be easily deterministically defined. The main aim of the component failure analysis is to add enough spare capacity to the system that will enable continuous provision of services with given standards even when failure occurs. Spare capacities are supplied either through adding of new components to a network layout, so called back up paths, or through the increase of network capacities. The focus in this study is on the identification of the minimum cost systems spare capacities that can secure system functioning under some predefined component failure scenarios, since the question of the existence of the back up diagram in a network layout, has been already addressed with a similar methodology by Ostfeld and Shamir (1996) and Ostfeld (2005).

For the performance failure analysis [4] the Stochastic Design approach is convinient alternative to incorporate probabilistically defined parameters into the design analysis. Since parameter's variability and uncertainty arise from socioeconomic (e.g. changes in water consumption, development of new water use technologies, etc.) and natural (e.g. changes in river flows,

[3] analysis of a system under conditions of a failure of an individual component or exposition to an extreme stress such as fire fighting

[4] analysis of a system under conditions of input parameters deviation from their measured, calculated or projected values due to their natural variability or uncertainty connected with their determination

corrosion, deposition, etc.) conditions as well as from our non-ability to measure the current conditions or predict the future ones with certainty. The range of acceptable parameters' variability and uncertainty is a subjective category that depends on a risk perception of decision maker. Therefore the parameter deviation range will be divided into classes of 1 %, 5 %, 10 % and so on of the total possible deviation, that correspond to different decision makers' risk acceptance levels. These levels, called tresholds, basically represent recognised level of the parameter uncertainty and variability by the decision makers and correspond to the percentual deviations from the predicted parameter values.

Uncertainty in water resources may result from the natural complexity and variability of hydrological systems and processes or from the unpredictable changes of human and society behaviour itself (Bogardi and Kundzewicz, 2002). These two types of uncertainty can be appointed to the water demands (loads to the system), water availability (resistance of a system) and the parameters of the system itself.

Traditional design is based on the premise that the system‘s resistance r has to sustain for all predefined load conditions s satisfying number of codified performance criteria, so called standards. This allows for the forward going determination of the system structure by gradually increasing system capacities, for each failure scenario, until the standard satisfactory performance is reached. The performance of the system is calculated from the function $f(s,r)$ that tests the systems resistance for every loading condition. Finally, in order to account for uncertainties the capacities of the obtained system structures are increased for the standard safety factors (left graph in Figure 2.4).

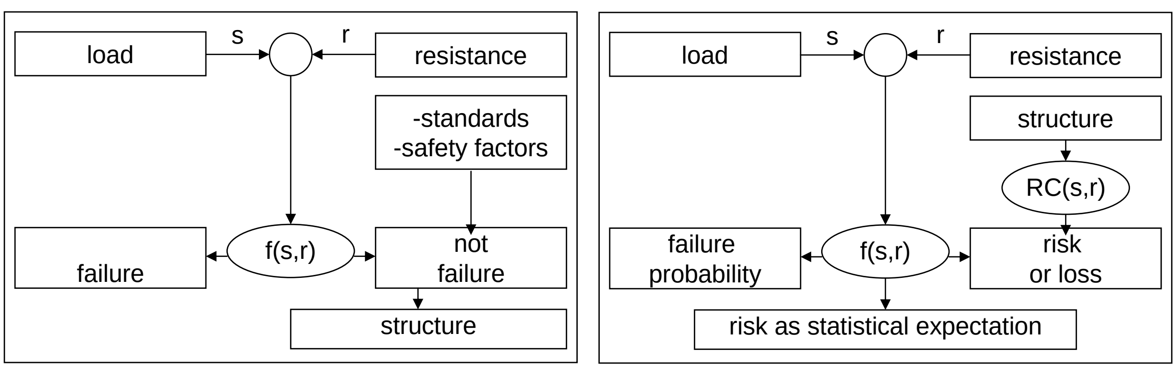


Figure 2.4.: Traditional and stochastic [source: Plate, 2000] design approaches

In contrast, the Stochastic Design does not assume deterministic system performance criteria but instead allows for a flexible definition of the satisfactory performance of a system according to users' or decision makers' risk acceptance. As presented by Plate (2000) and illustrated on the right graph in Figure 2.4 for every suggested system configuration, instead of safety factors, decision makers' risk-cost functions $RC(s,r)$ are used to accept or reject the system with a failure probability P_F. The failure probability $P_F = \int f(s,r)ds$ is obtained as a total probability of failures of a system performances $f(s,r)$ for each suggested system resistance r on which a range of probabilistically defined loads s is applied. The risk-cost functions do not necessarily have to depict the economic costs connected with some damage but may also be the costs of the low system performance, loss of good business reputation or potential customers, etc. An example of the risk-cost function is already given as risk tolerance-reliability function in Figure 2.3. The total accepted risk by adopting of some system configuration may be then expressed as the statistical expectation of the total costs of all expected failures for defined loading conditions:

$$RI = \int RC(s, r) f(s, r) ds \tag{2.1}$$

Such calculated total risk presents a basis for the selection among different options based on the decision maker's individual risk tolerance.

There are some other ways to substitute for a deterministic definition of uncertain parameters. In last few decades, one of the most often used are Fuzzy Sets (Zadeh, 1965, 1978). Among wide range of applications that may be found in literature (see Zimmermann, 1985, Bárdossy et al., 1983 and Bárdossy and Duckstein, 1995), some are specifically concerned with problems of water supply systems (Bogardi et al., 1987; Bárdossy and Duckstein, 1995; Vamvakeridou-Lyroudia et al., 2005). Aiming at the development of as simple as possible methodology, the probabilistic definition of uncertainty is adopted. Furthermore, the above presented concept for the risk assessment can be easily accommodated for fuzzy or in some other way defined input parameters or resistance criteria.

2.4. Management and Analysis of Water Supply Systems

Keeping in mind the complexity of water supply systems, their specific position between nature and society and their vital importance for the further society development, it is more than obvious that water planners, designers and managers need "help" to manage them. If the **System Analysis** is defined as a methodology to represent a real system by the means of mathematical equations and statements in order to "aid engineers, planners, economists and the public to sort through the myriad of schemes which are and could be proposed" (Loucks et al., 1981), it is not a wonder that this methodology has found many outstanding applications in the area of management of water resources (Maass et al., 1962b; Hall and Dracup, 1970; Haimes, 1977; Loucks et al., 1981; Haimes, 1984; Hipel and McLeod, 1992). But before a more detailed revision of the application of the System Analysis in the management of water supply systems, it is necessary to distinguish among main management stages that occur during the life cycle of a water supply system. The most often used approach is a hierarchical, suggested by Jamieson (1981), that distinguishes among planning, design and operation stage (Figure 2.5.).

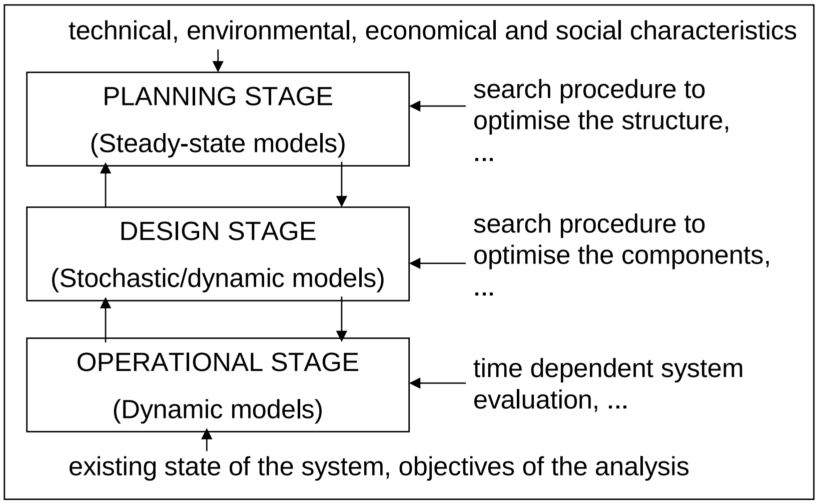


Figure 2.5.: Hierarchical approach to the management of water supply systems (Jamieson, 1981)

It is important to notice that the analysis is here understand as a "search procedure to optimize a system" where: a) the planning stage focuses on the systems structure, investment costs and development of the resources, b) the design stage searches for a minimum cost components that will satisfy required system quality and c) the operation stage aims to minimize systems operation costs, develop strategies for better maintenance, and tries to improve systems performances. In recent years the rehabilitation stage [5] gains an increasing importance but due to the generally similar aims as in design and operation stage it will not be separately considered in this study.

2.4.1. System Analysis in Planning of Water Supply Systems

An extensive review of water resource planning studies can be found in literature such as Singh (1981); Loucks et al. (1985); Viessman and Welty (1985); Wilson (1999); Yurdusev (2002). Still, for the purpose of better understanding of the proposed methodology the main development phases in application of the System Analysis in water supply systems planning are shortly presented.

The initial approach was to develop alternative water supply strategies was based on engineering logic and calculations among which is then selected mainly by evaluating their monetary costs and benefits. But already in late 1950's it was realized that many objectives of water resource planning analysis, such as increase in social benefits, recreational use, amenity value and many others, are hard to express in monetary terms. The Harvard Water Program (Maass et al., 1962a) is usually regarded as the starting point for the implementation of the System Analysis into water resources planning. Shortly after, O'Neill (1972) formulated the specific problem of water supply systems capacity expansion for the central area of South-east England as a mixed-integer programming problem. The objective was to identify

[5] upgrade and improvement of an already existing system

the minimum cost capital and operation development scenario by transferring the water resources from different potential sources with pre-specified yields to the demand centres with predefined marginal demands. At about the same time Butcher et al. (1969) used a dynamic programming model to determine the "optimal" construction sequence of additional system capacity to meet increasing demand. This model used the cost per unit supply available from each water source to differentiate among sources and was able to account for the effects of interest rate. Later on, it was modified by Esogbue and Morin (1971) to allow more general selection and sequencing of available expansion capacities.

Once set up as a minimum cost optimization problem, various system analysis techniques found their way in the planning studies of water supply systems. One of the most cited studies is the North Atlantic Regional Water Resource Study (Haimes, 1977; Cohon, 1978) that used the Linear Programming technique to allocate available resources to water demands. Since the Linear Programming is applicable only to problems that have linear dependencies among parameters, in 1980's various other mathematical programming techniques have been tried. de Monsabert et al. (1982) and Gorelick et al. (1984) tried with the Non-linear Programming but as identified by McKinney and Lin (1994) this technique is not able to handle interdependency among parameters and may have difficulties in determination of the gradients for highly non-linear dependencies. The Goal Programming technique, such as in Rajabi et al. (1999), suffered from often too large sets of possible system states and is therefore more convenient for nested problems with sequential decisions (Vink and Schot, 2002). These as well as many other techniques, based on the evaluation of gradients, tend to end up in local optima and are not convenient for discrete problems with many near optimal solutions (Dandy et al., 1996b).

In 1990's it has become clear that the exact mathematical programming techniques are computationally too demanding for complex optimization problems, and approximate techniques come into the play. Among them the Genetic Algorithms turned out to be the most often used one. Dandy and Connarty (1995); Dandy et al. (1996b) introduced this approach to project sizing and scheduling of different dam combinations and sizes while Vink and Schot (2002) used it for the determination of optimal production strategies from different groundwater sources. Another often used robust heuristic technique is Simulated Annealing. Ejeta and Mays (2005) used this approach for development of optimal timing of the capacity expansion of water supply conveyance and identification of optimal water allocation policy. Although such models have proved their value for many theoretical problems their application in practice is still waiting behind. The lack of good conceptual representation of the systems, or the one that does not coincide with the user conceptualization, may be one of the biggest reasons for that (Loucks et al., 1985; Walski, 2001).

Already in 1970's it was recognized that the consideration of only economic criteria does not suit to the complex multi-objective aims of the water supply planning analysis. Lawson (1974) tried to upgrade the model of O'Neill (1972) for considering environmental quality by omitting sources that are environmentally sensitive. Similarly Page (1984) developed an iterative procedure for allocating water transfers to meet water demands at five-year intervals by constraining of the environmentally sensitive sources. Another example of treating environmental issues as constraints to the, in this case, Transportation-type Programming approach is introduced by Stephenson (1982). Several optimization models focused on the incorporation of social trends and preferences into water supply system management. Lund (1987) used the Sequential Linear Programming method to evaluate and schedule water conservation measures that minimize system costs by avoiding or deferring capacity expansion while Rubenstein and Ortolano (1984) used the Dynamic Programming to design demand management option that supplements limited available water sources. Among models that used the Decomposition Approach to address the environmental impacts and socioeconomic effects together, the Wu (1995) and Kirshen et al. (1995) are among the most famous ones. Wu (1995) developed a separated module (Regional Model for Impact Assessment) in order to report on the state of physical quantities and socioeconomic quantities for different alternative development scenarios. Similarly, Kirshen et al. (1995) coupled modules for the evaluation of environmental, social and cultural impacts with water allocation among sources, demands and treatment facilities. This model, as well as some others such as Watkins et al. (2004); Yamout and El-Fadel (2005) is further developed with the aim to cover a full range of issues and uncertainties faced by water planners, including those related to climate, watershed condition, anticipated demand, ecosystem needs, regulatory climate, operation objectives and others. Such complex and sophisticated models are meant for governmental or national level water resources management and are not providing a practical solution for the planning and development of a single water supply system. In addition, they demand a very large amount of data and are quite cumbersome for practical use.

In parallel to the increasing awareness of the importance of environmental and social aspects in water resources management, a rapid progress in information technologies enabled the use of "interactive computer programs that utilize analytical methods, such as decision analysis, optimization algorithms, program scheduling routines, and so on, for developing models to help decision makers formulate alternatives, analyse their impacts, and interpret and select appropriate options for implementation" (Adelman, 1992). From this definition it is more than obvious that such models, often referred as Decision Support Systems (DSS), are a very complex aggregation of data processing tools (databases, statistical analysis software, etc.), simulation and optimization models (representation of process and creation of optimal system alternatives), and expert systems for the evaluation of alternative's effects and guidance of decision makers during the evaluation and selection of final plans. Loucks and da Costa (1991) give an excellent review of the application of DSS prior to the 1990's while the review of some of the numerous latter DSS models can be found in: Watkins and McKinney (1995), Ejeta and Mays (1998), AWRA (2001) and Geertman and Stillwell (2003). As far as the water supply planning in specific is concerned after development of numerous integrated ground and surface water bodies and water supply systems simulation-optimization models, such as in Nishikawa (1998); Belaineh et al. (1999); Srinivasan et al. (1999); Yang et al. (2000); Ito et al. (2001); Vink and Schot (2002) in recent years the researches focused on the better integration of primary issues such as water availability (Luketina and Bender, 2002), water demand (Hopkins et al., 2004) or institutional constraints (Ejeta et al., 2004). As a consequence there is an evident trend to reduce the complexity of the models in order to make them more practical and promote their greater use. Although also an agglomeration of quite a few sub-models, the CALVIN model (Draper et al., 2003) presents a good example for pragmatical approach in evaluating various benefits and costs of water provision and is based on very simple benefit and cost functions. In addition the model uses the network representation of water supply systems that makes it more understandable for potential users and more computationally effective. But the fact that it uses piecewise linear approximation of the cost and benefit functions theoretically hinders its usefulness for non-linear and concave problems.

Finally it can be concluded that the need for methods that are, on one side, based on easily understandable concepts and techniques and, on the other side, able to deal with complex multi-objective water supply planning problems still exists. Furthermore the need for the integration of economic, environmental and social objectives in the development of water supply strategies and the necessity for the transparent creation of a broad range of alternative water supply planning options in order to better support multi-objective decision making, are identified as the main priorities of the future research. The development of the methodology that is based on some simple mathematical representation and is able to integrate main technical, environmental and socioeconomic aspects of importance into one unique framework for the identification of the multi-objective water supply planning options, is the main attention of this study.

2.4.2. System Analysis in Design of Water Supply Systems

Since 1960's the optimization of water distribution networks has been one of the most heavily researched areas. Very comprehensive reviews can be found in: Walski (1985b); Goulter (1987); Walters (1988); Subramanian (1999); Lansey (2000). In 1980's Walski (1985a) and Goulter (1987) were predicting that the state-of-the-art optimization models of that time, will soon find their widespread use in practice. Although these models showed a certain degree of robustness and proved their capabilities of handling relatively complicated design problems in the famous "Battle of Network Models" (Walski et al., 1987), one decade later, the same authors (Goulter, 1992; Walski, 1995) were busy trying to identify the reasons why such predictions did not came true.

The first models for the water distribution network design (Karmeli et al., 1968; Schaake and Lai, 1969) were developed for branched networks and even though Swamee et al. (1973) proved the optimality of branched network for a single demand pattern, networks with no built-in redundancy were of no interest for practice. Although from an engineering intuitive point of view, the loops have been already for a long time recognized as a "best practice" way to bring redundancy into the system, for the modellers, the loops have brought significant complexity into the algorithms. While in a branched system a given demand pattern uniquely defines the flows in the network, in a looped system there is a very large number of flow combinations that can meet a specified demand pattern (Goulter, 1992). Only in late 1970s the researchers have managed to solve the distribution network design problem by decomposing it into an optimization part, which searches for minimum cost design parameters, and a simulation part that calculates network hydraulic properties for one design configuration (Alperovits and Shamir, 1977; Bhave, 1978; Quindry et al., 1981; Rowell and Barnes, 1982). As identified by Templeman (1982), these first looped network designs were "implicitly branched". They were made by cross connecting optimized branched systems and, as noted in the same work, do need a sufficient number of simulations with different demand patterns or component failure scenarios to increase capacities on all alternative paths. In order to improve the procedure for finding an optimal solution and better address some inherent system properties such as redundancy, reliability or uncertainty of input parameters, researchers have tested different approaches such as deterministic, stochastic, heuristic, entropy based and various types of their combinations.

One of the most important deterministic network optimization works is the Linear Programming Gradient Method of Alperovits and Shamir (1977), which firstly formulate hydraulic loops for each source-demand node path and then modify the flow distribution based on the gradient of total costs with respect to such a change. This method improved by Quindry et al. (1981) as well as similar formulations based on the Linear Programming techniques from Lansey and Mays (1985), Fujiwara et al. (1987), and Kessler and Shamir (1989) or the Sherali and Smith (1993) approach with design capacities as optimization variables instead of flows, suffer of finding only a local optima, since this is an inherent property of gradient based searches. Moreover, starting with Chiplunkar et al. (1986), many researchers have tried to use the Non-linear Programming technique but in addition to the local optima problem (Gupta et al., 1999), as identified by Cunha and Sousa (1999), the conversion of discrete market available pipe diameters to continuous variable additionally influence slow convergence of the solution technique. Although such results significantly enforced the use of stochastic procedures it must be noted that the approach proposed by Eiger et al. (1994) is often identified as first global solution to the network design problem. This algorithm employs the Branch and Bound procedure to control the production of an improving sequence of local solutions, the hydraulic consistency is provided via enumeration of all possible basic loops and source-demand node paths while the prescribed tolerance between the global lower bound produced by solving a dual problem and the best funded value define stopping criteria. Sherali and Smith (1997) used the Tight Linear Programming relaxations in order to compute lower bound and also embedded their Reformulation-Linearisation technique in a Branch and Bound scheme. Although these algorithms and some of their later improvements (Sherali et al., 1998, 2001) solved some of the test problems for network design to the global optimality the computational demands and models complexity were still too high to be used by practitioners.

Being a non-convex problem with discrete decision variables and a large number of local optima, the network design problem has been in recent years frequently addressed by stochastic and heuristic optimization techniques. The stochastic procedures are mainly used to address the uncertainty of the input parameters and heuristic procedures to advance the optimization process. Capability of simultaneous dealing with a set of discrete points from decision variable space, flexible formulation of objective functions and ease to escape local optima present some of the main advantages of heuristic methods. These methods are very computationally demanding and the randomness of the funded solution give no possibility to prove whether it is a true global optimum or not. The Genetic Algorithms have been the most often used heuristic optimization technique (Simpson et al., 1994; Dandy et al., 1996a; Savić and Walters, 1997; Abebe and Solomatine, 1998; Kapelan, 2003; Tolson et al., 2004; Babayan et al., 2004; Prasad and Park, 2004; Farmani et al., 2005; Giustolisi and Mastrorilli, 2005), but the Simulated Annealing (Loganathan et al., 1995; Cunha and Sousa, 1999), the Ant Colony Optimization (Maier et al., 2003), the Shuffled Frog Leaping (Eusuff and

Lansey, 2003), the Shuffled Complex Evolution (Liong and Atiquzzaman, 2004) and others have been used as well. Giustolisi and Mastrorilli (2005) integrated the Genetic Algorithm optimization technique with variance reduction Monte Carlo sampling technique, called the Latin Hypercube, to allow fast identification of a set of near optimal solutions with accurate sampling of probability functions related to the uncertainty of the design conditions. Although these optimisation techniques showed excellent performances in solution of many very complex theoretical water supply design studies, many of them are still too complex for an average engineering level of knowledge to be more often applicable in practice.

Having identified effective and robust optimization routines for the minimum cost network design problem, the researchers have realized that, in practice, "the optimal design of a water distribution network is a complex multiple objective process involving trade-off between the cost of the network and its reliability" (Xu and Goulter, 1999). In middle 1990's, Goulter (1995) and Mays (1996a) have provided the most comprehensive review of the reliability analysis works and have stated that the reliability issue is one of the most challenging in the field of water supply engineering. Two decades later it is still an open research area and tempts for new solutions.

From the point of view of the component failure analysis (failure of individual system components) already Rowell and Barnes (1982) develop a procedure to interconnect pipes in order to maintain the required level of services. Later on Goulter and Morgan (1983) incorporated a feedback mechanism and even expanded it with a heuristic search procedure (Morgan and Goulter, 1985). Lansey and Mays (1989) further advanced this procedure to enable simulation of multiple loading conditions. Many other works from the field of the component failure analysis have been based on the Path Enumeration Methods (Tung, 1996a) among which the Cut-set Analysis [6] and the Tie-set Analysis [7] are the most often used ones. Shamir and Howard (1985); Morgan and Goulter (1985); Tung (1985); Goulter and Coals (1986); Shamsi and Quimpo (1988); Mays (1989a); Bouchart et al. (1989) used these techniques but as identified by Khomsi et al. (1996) their applicability to water networks is rather limited. Firstly due to the quite unrealistic condition that all pipes in a minimum cut set would be in a failure state at the same time, secondly due to the extensive computation needed for the identification of all minimum cut sets and thirdly due to the fact that the use of the basic cut set methods do not incorporate any of the hydraulic conditions which may govern the flow in a network (the supply to a node may fail completely due to pressure insufficiency without being entirely isolate by broken pipes). The authors themselves use a simple stochastic model to simulate pipe breakages and insufficient pipe capacities but not for a pre-processing and evaluation of demands uncertainty but for a post-processing in order to test the reliability of a water supply system.

For further development of the component failure analysis the terms: **reachability** [8] and **connectivity** [9] defined by Wagner et al. (1986, 1988a,b) were of crucial importance. These terms come from the Conditional Probability Reliability Procedures (Tung, 1996a) and many

[6] set of system components or modes of operation which, when failed, cause failure of the system

[7] set of system components arranged in series which, fails when any of its components or modes of operation fail

[8] probability that a given demand point is connected to at least one source

[9] probability that all demand points are connected to at least one source

researchers used them together with the Minimum Cut-set methods (Su et al., 1987; Quimpo and Shamsi, 1987, 1991). Although the latter was significantly improved by Quimpo and Wu (1997) to include hydraulic measures and capacities in the reliability measure, by Yang et al. (1996) to simplify the algorithm and by Shinstine et al. (2002) to implement the repair-ability of the components, the Minimum Cut-set approach still suffer from large computational demand needed to calculate path sets for each component or component states combination. Kessler et al. (1990) developed a much less computational demanding methodology, which even ensures a certain degree of redundancy, and extended it later together with Ormsbee and Kessler (1990) to include capacity constraints. Still due to the superficial interpretations that have not been adequately packaged for practical system-design environments these methodologies have been underutilized (Beecher et al., 1996).

Lansey et al. (1989) were the first to address the uncertainty in demands and they used a chance-constrained model to add demand uncertainties upon pressure and pipe roughness coefficient uncertainties. Bao and Mays (1990) used the Monte Carlo Simulation for the same purpose while Duan et al. (1990) used the Continuous-time Markov process to model the available capacity of pump stations. Many other works in demand variation and hydraulic failure analysis used probability theory or stochastic simulation to define, or constrain, uncertainty in demands and hydraulic network performances. These works ranged from simple analysis such as: supply demand quantities (Beim and Hobbs, 1988; Hobbs and Beim, 1988; Duan et al., 1990) and use of ratio of expected maximum total demand to total water demanded (Fujiwara and De Silva, 1990; Fujiwara and Tung, 1991), over use of the chance-constrained network design for limiting the shortages at nodes in comparison to demand values (Tung, 1985; Park and Liebman, 1993), use of assumed theoretical probability distribution functions of nodal demands and pipe roughness (Xu and Goulter, 1998, 1999), use of the First-order Reliability methods to assess the demands probabilities (Goulter and Coals, 1986; Goulter and Bouchart, 1990; Tolson et al., 2004), reformulation of the stochastic problem as the deterministic one using standard deviation as measure of the variability of demands (Babayan et al., 2003), to the representation the nodal demands as fuzzy numbers (Bhave and Gupta, 2004). In its analysis of the previous research Goulter (1992) identifies the work of Bouchart and Goulter (1991) as an interesting example of joint consideration of two failure modes (component and demand variation failure) but still states general difficulty of considering both phenomena simultaneously and disparity among models which are computationally suitable for inclusion in optimization frameworks and the ones with good network performance. This may well be the reason while in most of these models the network performance and optimization are still decomposed.

Another line of thought explored the concept of entropy, introduced by Templeman (1997), to assign most likely flows to alternative paths and incorporate redundancy in the optimization of water distribution systems (Awumah et al., 1990, 1991, 1992). Tanyimboh and Templeman (1993) suggested that flexible networks can be achieved through maximizing the entropy of flows and significantly reduced computational time by using estimates obtained by averaging the upper and lower bounds on reliability (Tanyimboh et al., 1997). Tanyimboh and Sheahan (2002) proposed the idea of minimum cost maximum entropy designs to identify good layouts of water distribution systems. Still the relationship between entropy and reliability has yet to be properly established.

Finally it is not to be forgotten that network reliability is in fact defined, or more specifically constrained, by the fundamental layout of a network (Goulter, 1987). At the same time, the shape of a network significantly effects the costs and improvements in reliability tend to degrade the minimum cost objective. In order to address the question of the network layout, the Graph Theory has been almost exclusively used. Furthermore, the connectivity and reachability of Wagner et al. (1986), were again a very important research milestones. Based on the Graph Theory, Ormsbee and Kessler (1990) developed an algorithm which identifies two independent paths to each demand node. Jacobs and Goulter (1989) investigated the use of a regular graph target (optimal reliability should be provided by an equal number of graph links or arcs incident on each node) but concluded that such an approach is not applicable to water supply systems due to the semi-branched structure of water distribution networks since peripheral nodes need fewer links incident upon them. Diba et al. (1995) presented a very interesting combination of the Directed Graph algorithm and the Linear Programming procedure for solving various large-scale water distribution problems. Although this model is primary developed to assist the planning process it can be further extended for design purposes. Based on a connectivity analysis of the network's entire topology, Ostfeld and Shamir (1996) introduced the concept of backups and recently, Ostfeld (2005) expanded it to produce the most flexible pair of operation and backup digraphs that yield first-level system redundancy (if one arc fails, a minimum of one path from at least one source to all consumers is retained). Although these works provide a very good basis and even propose some very practicable suggestions for the network layout design, it has to be noted that the connectivity/topology analysis has been one of the least researched areas in the water supply system design.

In the end, it is also important to refer to some of the works which specifically address the issue of multi-objectiveness in water supply system design and importance of the trade-off between costs and network performances. Already de Neufville (1970) in his cost-effectiveness analysis promoted the introduction of system specific objectives and alternative levels of performances rather than application of the Standards for civil engineering systems design. Furthermore, the same author recognizes the necessity to address the institutional, social and behavioural issues that may effect, or constrain, the system design. A very good overview of the application of multi-objective optimization can be found in Van Veldhuizen and Lamont (2000) and it seems that among many approaches for dealing with more-then-one objectives and criteria, the Pareto Dominance Criterion (Pareto, 1896) has been the primarily used one (Dandy et al., 1996a; Savić and Walters, 1997; Kapelan, 2003; Tolson et al., 2004; Babayan et al., 2004; Farmani et al., 2005; Giustolisi and Mastrorilli, 2005).

As it can be seen from the above, it is very hard to select among numerous offered approaches and methodologies and although all individual issues have been already treated, one simple and easily understandable method with clear representation of the system able to comprise the multi objective nature of the design process and integrate minimum cost solutions with the reliability issues, such as components failure and parameters' uncertainty is still to be found. Exactly the problem of the identification of the design solutions that provide for the optimal trade-off among system costs and its reliability is the main focus of the design analysis in this study. Furthermore, the development of the methodology able to encompass system investment and operation costs defined with various non-convex function and the system reliability assessment based on the component failure and the parameter uncertainty analysis is expected. Finally an approach that allows for the integration of the decision makers' perception of the needed performance and reliability of water supply systems based on their risk acceptance is adopted instead of the traditional design based on the engineering standards and codes.

2.4.3. System Analysis in Operation of Water Supply Systems

The two most obvious aims of the water supply systems operation are to control the hydraulic performance during operation and to minimise the economic expenditures of water supply provision. Since the economic expenditures are mainly made of operation and maintenance costs, the minimisation of these two is an imperative since the early times of conventional water supply systems. Furthermore, if known that even today in the UK for example, the electricity costs make approximately 10 % of the total operating expenditures of large water services companies and that the pumps consume more than 70 % out of these costs (Yates and Weybourne, 2001), then it is not a wonder that the identification of the minimum cost pumping operation policies is a very active area of research in last three decades. As in many other areas that needed computer support for the solution of complex and computationaly demanding problems, the System Analysis found many usefull applications in the water supply systems operation as well.

A very good overview of the research work in the water supply systems operation until 1993 can be found in Ormsbee and Lansey (1994). The authors define the operation policy (operation schedule) as the set of rules when a particular pump or group of pumps are to be turned on or off over a specified period of time and classify until that time available methods on the basis of their applicability, applied optimization method and nature of resulting policies. At that time the Linear Programming (Jowitt et al., 1989; Crawley and Dandy, 1993), the Mixed-integer Linear Programming (Little and McCrodden, 1989), the Dynamic Programming (DeMoyer and Horwitz, 1975; Sterling and Coulbeck, 1975; Sabet and Helweg, 1985; Zessler and Shamir, 1989; Ormsbee et al., 1989) and the Non-linear Programming (Chase and Ormsbee, 1989; Lansey and Zhong, 1990; Brion and Mays, 1991) were among the most often used methods. In addition to the problems with the linearisation of functions such as pump efficiency, the large computational demands, and the limitation to the smaller systems all these methods were exclusively searching for the optimisation of the only one objective, namely electric energy costs.

Lansey and Awumah (1994) directly related the number of times a pump is turned on and off over a given life cycle with the pump wear and enabled for the addition of the pump maintenance costs as the second objective that is to be minimised with the optimisation analysis. Although today the development of durable and high quality pump's material makes this dependency as questionable, the recognition of the importance of the inclusion of multiple objectives in the optimisation of the water supply systems operation significantly added to the problem complexity and favoured the application of the approximate methods instead of the analytic ones.

Two applications of the Knowledge-based selection (Fallside, 1988; Lannuzel and Ortolano,

1989) that combine a simulation model with a rule based expert optimisation system, provided some insight into the utility of the Expert System approach. Kansal et al. (2000) continued this approach and developed an expert system, called EXPLORE, for the management of the Seville City water supply system that achieve 25 % reduction of the energy costs.

Pezeshk and Helweg (1996) applied the Adaptive Search algorithm and Mäckle et al. (1995) applied the Genetic Algorithm for the optimisation of the pumping electric energy cost. Savić et al. (1997) proposed a hybridisation of the Genetic Algorithm with a local search method, in order to include pump maintenance costs and de Schaetzen (1998) included the system constraints by establishing penalties functions. Barán et al. (2005) proved that the Evolutionary Computations are a powerful tool to solve optimal pump-scheduling problems and successfully tested six different Multi-objective Evolutionary Algorithms on a problem with four objectives: electric energy cost, maintenance cost, maximum power peak and level variation in a reservoir. Maximum power peak, or maximum demand charge, is actually a penalty on a suggested pumping schedule if a certain pumping power of a system is exceeded. It comes from the fact that some electricity companies charge their big clients according to a reserved power and have expensive additional charge if this power is exceeded. Level variation represents the intention to satisfy minimum and maximum water levels in tanks and reservoirs as well as to recover the initial level by the end of the optimisation period. In many other studies this objective is model as a constraint.

McCormick and Powell (2003) also included the maximum demand charges (maximum power peak) in their optimisation of pumping schedules based on the Stochastic Dynamic program. Furthermore they modelled the variations in water demand as a discrete first-order Markov process and account for the transition probabilities of water demands based on the regression analysis in order to avoid the discrepancy of the optimal options accommodated to the time of the day dependent electricity costs and the options that avoid maximum demand charges. The same authors developed a two stage simulated annealing algorithm to efficiently produce optimal schedules that include pump switching and maximum demand charge objectives (McCormick and Powell, 2005). The method produces solutions that are within 1 % of the linear program-based solutions and can handle non-linear cost and hydraulic functions.

Nevertheless some of the problems already identified by Ormsbee and Lansey (1994) are still to be solved. The one that is going to be addressed in this study is the implication of design on operation, and vice versa. Even though it is obvious that the design of a water supply system will largely influence its operation, there has been amazingly little research that integrate these two. Farmani et al. (2005) applied the Multi-objective Evolutionary Algorithm for the identification of the pay-off characteristics between total cost and reliability of a water supply system where the design variables are the pipe rehabilitation decisions, tank sizing, tank location and pump operation schedules. Nevertheless the resilience index [10] and the minimum surplus head [11] that are adopted as the measures of the system reliability, are questionable. An attempt will be made to consider the optimisation of the water supply system operation already during the design stage. In particular, the system components that enable for more or less effective system operation, such as water tanks and reservoirs,

[10] the measure of the more power than required at each node

[11] the amount by which the minimum available head exceeds the minimum required head

will be considered together with the identification of the optimal pumping schedules. The trade-off among investment costs in storage facilities and operation costs of pump stations and the development of the model that can create alternative system configurations with different tanks positions and sizes and identify optimal pumping schedules for each of them is the main focus of the applied operation analysis. The final aim is the denomination of the Pareto-front of optimal system configurations and corresponding pumping polices that enable the selection of tank sizes and location which will provide for the most cost-effective operation of the water supply systems.

3. Methodology Development

The following chapter lays down the methodological foundations for the achievement of the stated study objectives. It introduces the adopted graph theory concept for the representation of water supply systems structure and function, and identifies a convenient mathematical formulation for the optimisation problem definition. This general formulation is accommodated for the integration of environmental and socioeconomic aspects as well as for the integrative analysis of fixed and variable impacts and effects. After discussion of the characteristics of the problem, the methods and algorithms suggested for the problem solution are explained and sorted out into unique methodology for the multi-objective and risk-based analysis and optimisation of water supply systems.

3.1. Representation of Water Supply Systems and Objectives of the Analysis

Keeping in mind that the essential role of water supply systems is to redistribute water resources in temporal and spatial scales from the times and places where there are available to the ones where there are needed, these systems can be intuitively seen as connecting elements among source and demand points. The source points determine quantity and quality of available water, the demand points the characteristics of needed water, while the elements in between facilitate water acquisition, treatment, storage and delivery. For such spatially distributed systems a Graph theory provides a very convenient way for the representation of system's structure, properties and function. Furthermore, it offers numerous algorithms for the solution of many problems defined on systems that consist, or may be represented as sets of nodes and arcs. Some basic definitions, necessary for the representation of water supply systems in graph theory terms, with notation as in Hartmann and Rieger (2002), follow.

3.1.1. Water Supply System's Structure

A **graph** G is an ordered pair $G = (N, A)$ where N is a set of **nodes** or **vertices** $n_i \in N$ and A a set of pairs $a_{i,j} \in A$, called **arcs** or **edges**, such that $A \subset N \times N$. A water supply system can be represented as a graph consisting of three different type of nodes: n_s - *origin* or *source nodes* representing the supply points of the system, n_d - *destination* or *sink nodes* representing the demand points, n_t - *transshipment* or *intermediate nodes* representing storage, treatment and transport facilities, and $a_{i,j}$ - *arcs* representing all elements that provide and control flow of water and the distribution of pressure such as pipes, tunnels, open channels, valves and pump facilities.

If the arcs a_{ij} are ordered pairs, then G is called **directed graph** or **diagraph**, otherwise G is called **undirected** and then a_{ij} and a_{ji} denote the same arc. Since the directed graphs are much easier to deal with and often have simpler algorithms, water supply systems will be represented as directed graphs in this study. In addition, by allowing the positive and negative values for flow on directed arcs, the variable direction of water flow in the distribution part of water supply systems, can be just as well presented on directed as on undirected graphs. For both directed and undirected graphs if an arc a_{ij} exists then n_j is **neighbour** of n_i (and vice versa), and n_i and n_j are **adjacent** to each other while arc a_{ij} is **incident** to these two nodes. The **degree** of a node $d(n_i)$ is the cardinality[1] of the set of its neighbours $d(n_i) = \mid \{a_{ij} \mid (n_i, n_j) \in N \vee (n_j, n_i) \in N\} \mid$. In directed graphs the arc a_{ij} can be referred as **outgoing** from n_i and **ingoing** to n_j. The **outdegree** $do(n_i)$ is then the number of outward directed arcs from a given node and **indegree** $di(n_i)$ the number of inward directed arcs to a node.

Since water supply systems are not just any collection of arcs among supply and demand nodes, but instead an ordered set of arcs that transport water from a specific supply node, first to the treatment facility and than further to the predefined demand nodes, a more specific graph theory term, so called path, is introduced. A **path** π is a sequence of nodes $n_1, n_2, ..., n_k$ which are connected by arcs a_{ii+1} such that $\forall i = 1, 2, ..., k-1$. For directed graphs, a path is said to be **forward** π^+ if its arcs are aligned in their forward direction and **backward** π^- if its arcs are aligned in their backward direction.

In addition, for the water supply systems layout and component failure analysis it is important to define few more terms. A set of nodes is called **connected component** if it contains only nodes, where from each node a path to each other node of the set exists. Consequently a graph that fits entirely into only one connected component is called **connected**. For digraphs this term is distinguished on **strongly connected** if there is a directed path between every pair of nodes and **weakly connected** if there is an undirected path between any pair of nodes (Skiena, 1990). Obviously a water supply system has to be at least *weakly connected*. If the minimum number of arcs whose removal would disconnect the graph is k, then it is called **k-edge-connected** or **k-connected** graph and **cut-set** is a set of arcs, which if removed, disconnects the graph. Final, graph theory term introduced here is **reachability** or the existence of a path, of any length, from one node to some another node. It served as a basis for the definition of the two very important terms for the reliability analysis of water distribution networks. Namely, *connectivity of demand nodes* as the probability that all demand nodes are connected to at least one source and *reachability of a demand node* as the probability that a given demand node is connected to at least one source (Wagner et al., 1986).

[1] relative notion of the size of a set which does not rely on number. For instance, two sets may each have an infinite number of elements, but one may have a greater cardinality (PlanetMath.Org, 2006)

In order to efficiently manage graphs (store their structure, search through them or reorder them according to some attribute) an additional term is introduced. A **tree** stands for connected graphs that can be redrawn in the following way:

- all nodes are arranged in levels $l = 0, 1, ..., h$,
- arcs exists only between nodes of adjacent levels, where l - *father node* and $l+1$ - *son node*,
- on level zero there is only one node, called **root**.

If the tree includes all the nodes of graph G it is called a **spanning tree**. In this study, a special type of tree called **list** (tree with exactly one *father* and only one *son* node) is used to store the structure of a water supply system. This is achieved by storing not only the node's and arc's identification numbers but also the information about the structure among the them (e.g. neighbor, adjacency, incidence, etc.) in, so called, *pointer-lists*. For example, the *origin-pointer-list* contains indexes of the lowest numbered arc originating from node n_i and the *terminal-pointer-list* contains indexes of the first entry in the list of arcs ordered by increasing terminal node that terminates at node n_i. These two provide a very efficient way to identify all arcs that originate or terminate at some node n_i Jensen (1980). Furthermore, pointer lists can be ordered according to the properties of the elements such as distance, cost, free capacity, etc. in order to better support search algorithms, such as for the identification of the shortest (e.g. minimum distance, minimum cost) or augmenting (i.e. maximum free flow capacity) paths. This, often called *arc oriented representation*, provides significant savings in terms of computer storage for spare networks such as water supply systems, while the use of pointer lists can significantly decrease the computing time (Jensen, 1980).

In the form of *nodes-list* and *arcs-list*, the structure of the water supply system with total N nodes and M arcs, can be represented as:

$$\begin{aligned} N &= [n_1, n_2, ..., n_N] \\ A &= [a_1, a_3, ..., a_M] \end{aligned} \tag{3.1}$$

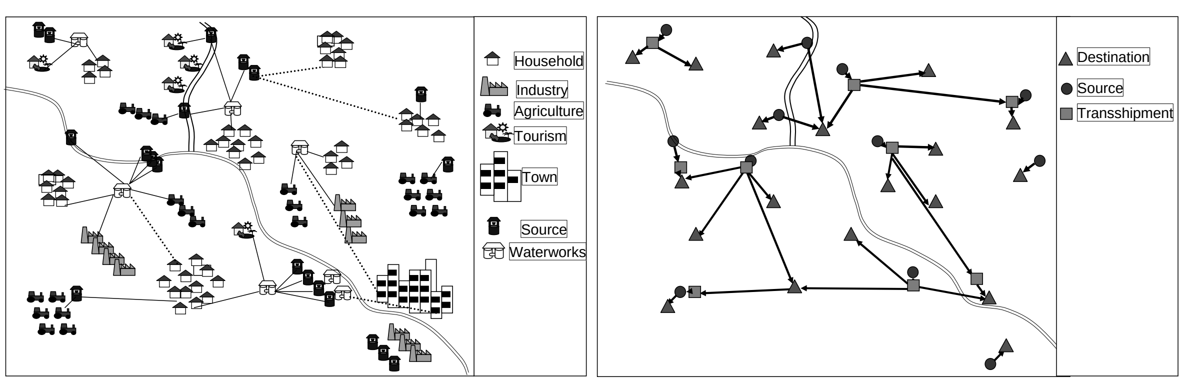


Figure 3.1.: Network representation of water supply systems

In Figure 3.1. it is schematically presented how a complex "real life system" from the left picture can be substituted with a graph of nodes and arcs among them. Obviously, such kind of simplifications have sense and value only for large water supply systems (mainly regional ones) that have many different sources and delivery points, consist of many transport, storage and treatment facilities and in which flow of water may take many different paths that can not be easily identified and analysed by manual calculations.

3.1.2. Water Supply System's Function

In addition to the possibility to represent the elements and the structure of a water supply system, the Graph Theory enables for the representation of the element's properties. Graph's arcs and nodes have attributes that may correspond to the properties of a real system, such as capacities, lengths, costs, etc.. Due to the frequent use, graphs whose arcs have capacity as an attribute have their own name, *networks.*

The exact definition of a network, as in Hartmann and Rieger (2002), that is based on the assignment of some arbitrary functions on arcs or nodes, called **labelling**, $f_a : A \rightarrow Q$ from arcs to rational numbers and $f_n : N \rightarrow S$ from nodes to arbitrary set reads as "a **network** is a tuple, or an ordered set of n elements, $G_n = (G, \kappa, n_s, n_d, n_t)$ where:

- $G = (N, A)$ is a directed graph without arcs of the form a_{ii},
- $\kappa : A \rightarrow R_0^+$ is a positive labelling of arc capacities and $\kappa(a_{ij}) = 0$ if $a_{ij} \notin A$,
- $n_s \in N$ is a node called source with no incoming $di(n_s) = 0$ arcs,
- $n_d \in N$ is a node called destination with no outgoing $do(n_d) = 0$ arcs,
- $n_t \in N$ is a node called transshipment with incoming $di(n_t) > 0$ and outgoing $do(n_t) > 0$ arcs".

Flow of water over a network is an attribute of the arcs. It is usually denoted as x_{ij} and represents the quantity of water flowing through an arc a_{ij} in a period of time. For a given network $G_n = (G, \kappa, n_s, n_d, n_t)$ a set of flows on all arcs $\mathbf{x} = \{x_{ij} \mid a_{ij} \in A\}$ is referred as *flow vector* or *flow pattern.* In addition, according to the previous definition of the path, a *path flow* is a vector that corresponds to sending a positive amount of flow along arcs of a path, or more precisely, it is a flow vector $\mathbf{x}$ with components of the form (Bertsekas, 1998):

$$x_{ij} = \begin{cases} a & if & a_{ij} \in \pi^+ \\ -a & if & a_{ij} \in \pi^- \\ 0 & otherwise & \end{cases} \tag{3.2}$$

where a is a positive scalar and π^+ and π^- are the forward and backward paths. As proved by Bertsekas (1998) "any flow vector can be decomposed into a set of conforming paths", where a path flow x^π *conforms* to a flow vector $\mathbf{x}$ if it carries flow in the forward direction ($x_{ij} > 0$ for all forward arcs and $x_{ij} < 0$ for all backward arcs on the path π) and if a forward path have a source and destination node as start and end node, respecitively. This simple proposition can be extremely useful in the analysis of systems such as water supply systems, where the main aim is exactly to analyse the water paths from source to destination nodes.

Before further proceeding into the definition of the governing equations and their constraints, the following simplifications can be made. Firstly, although different commodities (e.g. raw water, treated water) may be transported over a water supply network, there is no need to consider multi-commodity flows on water supply networks, since there are no arcs at which two different commodities flow at the same time. And secondly, the conservation of flow in arcs will be assumed and the pipe losses will be then additionally accounted for.

Under the assumptions that water in a water supply network is an incompressible fluid [2] and that the temperature differences are small [3], the continuity and momentum equations are sufficient to determine the velocities and pressures in a water supply network.

The **momentum equation** basically states that for any small, fixed control volume of fluid, the rate of change of momentum must equal the sum of any external forces acting on the control volume. In order to simplify the writing of momentum equation, for a constant diameter pipe, the parameters that influence the forces of fluid weight, pressure and friction are often represented by, so called, pipe characteristics r_{ij}:

$$r_{ij} = C_{\tau_{ij}} \frac{L_{ij}}{2gA_{ij}^3} \quad \forall a_{ij} \tag{3.3}$$

where L_{ij} - length of a pipe, A_{ij} - cross section area of a pipe and $C_{\tau_{ij}}$ - coefficient of tangential friction (friction coefficient). Since the change of flow velocity in constant diameter pipe can be approximated to zero for incompressible fluid, the change of momentum in control volume is equal zero and the momentum equation is reduced to the summation of the forces and may be written as in (Ivetić, 1996):

$$\begin{aligned} &\Pi_i - \Pi_j = r_{ij} x_{ij}^2 && \forall a_{ij} \quad \textit{or alternatively} \\ &\Pi_i - \Pi_j = r_{ij} x_{ij} \left| x_{ij} \right| && \forall a_{ij} \end{aligned} \tag{3.4}$$

where Π_i - is a head at node i and represents the sum of kinetic (p_i-pressure) and potential energy (z_i-elevation over some reference point, usually sea level). *Lower-*λ_{ij} and *upper-*κ_{ij} *capacity bounds* of the network arcs represent constraints that have to be respected.

$$\begin{aligned} x_{ij} &\le \kappa_{ij} \quad \forall a_{ij} \in A \\ x_{ij} &\ge \lambda_{ij} \quad \forall a_{ij} \in A \end{aligned} \tag{3.5}$$

where lower bound λ_{ij} is usually equal 0. For directed networks a negative flow constraint, or so called *skew symmetry* constraint, must be fulfilled too.

$$x_{ij} = -x_{ji} \quad \forall a_{ij} \in A \tag{3.6}$$

[2]has constant density
[3]small enough that no heat flux occur

In addition to water conservation in arcs, water has to be conserved on nodes too. Sum of inflows must be equal to sum of outflows except on source and destination nodes where it equals external flow. The **continuity equation** can be written as (Ivetić, 1996):

$$\begin{aligned} &\sum_{n_j} x_{ij} + b_j = 0 && \forall n_j \quad \textit{or alternatively} \\ &\sum_{n_j : a_{ij} \in A} x_{ij} - \sum_{n_j : a_{ji} \in A} x_{ji} + b_j = 0 && \forall n_j \end{aligned} \tag{3.7}$$

where b_j represent the external flow which comes in or leaves the system. For source nodes $b_j > 0$, for destination nodes $b_j < 0$ and for transshipment nodes $b_j = 0$. Consequently, these values have to satisfy maximum available and minimum demanded water amounts at source and demand nodes, respectively:

$$\begin{aligned} b_j &\leq S_{max_j} && \forall n_{s_j} \in N \\ b_j &\geq D_{min_j} && \forall n_{d_j} \in N \end{aligned} \tag{3.8}$$

Finally, the total energy (Π_i), or its kinetic component (p_i-pressure) is mainly bounded by engineering standards for satisfaction of users services and network safety which represents the last constraint to the above defined equations:

$$p_{min_j} \leq p_j \leq p_{max_j} \quad \forall n_j \in N \tag{3.9}$$

It is important to mention that the term r_{ij}*-pipe characteristics* introduced in Equation 3.4 contains the friction coefficient expressed as tangential tension coefficient C_τ. There are many other ways to express the friction losses in the momentum equation and the three most often used will be mentioned here.

For example the Darcy-Weissbach friction coefficient λ, where $\lambda = 4C_\tau$ and for a circular pipe ($A/O = D/4$) transforms the pipe characteristics to the:

$$r_{ij} = \lambda_{ij} \frac{8L_{ij}}{g\pi^2 D_{ij}^5} \tag{3.10}$$

where D_{ij} is a pipe diameter. Another very popular expression of the pipe characteristics is a Hazen-Williams formula. Its friction coefficient C is not a function of velocity, is applicable only to the water flows at ordinary temperatures ($4 - 20°C$) and has different flow exponent in the momentum equation:

$$\Pi_i - \Pi_j = r_{ij} x_{ij}^{1.852} \quad \textit{and} \quad r_{ij} = k4.727 \frac{L_{ij}}{C_{ij}^{1.852} D_{ij}^{4.871}} \tag{3.11}$$

where k is a unit conversion factor: $k = 1.318$ for English and $k = 0.85$ for SI units. The third most often used, is the Chezy-Manning friction coefficient n, for which the pipe characteristics becomes:

$$r_{ij} = k4.66n^2 \frac{L_{ij}}{D_{ij}^{5.33}} \tag{3.12}$$

The selection of one of these equation depends on the available network data and all of them will be integrated in the latter developed models.

3.1.3. Formulation of the Optimization Problem

After showing how to mathematically define the structure and the most important characteristics of water supply systems, it is necessary to mathematically formulate the aims and objectives of a water supply analysis. Since the most common understanding of the function of water supply systems is to transport water through a network in order to satisfy demands at the destination nodes form available supplies at the source nodes by providing a good quality service to the users with the minimum costs and negative effects, in the most general terms the aim of the water supply systems analyses can be stated as the cost minimisation of network flows, often called **Minimum Cost Network Flow** problem. Bertsekas (1998) defines this problem as: "search for a set of arc flows that minimize a given cost function, subject to the constraints that they produce a given divergence[4] vector and that they lie within some given bounds". Due to the functional dependency of flows and network properties, the identification of flows that give a minimum cost over a network can be used to identify the minimum cost layout, capacities, energy input, or some other network characteristic. This flexibility makes the Minimum Cost Network Flow problem one of the most often implemented optimization formulations in many engineering disciplines (Henley and Williams, 1973; Biggs et al., 1976; Harary, 1994).

The Minimum Cost Network Flow problem, defined on arcs, for water supply networks can be stated as:

$$min \quad z = \sum_{a_{ij} \in A} c_{ij} x_{ij} \tag{3.13}$$

subject to:

$$\begin{aligned} &\sum_{n_j : a_{ij} \in A} x_{ij} - \sum_{n_j : a_{ji} \in A} x_{ji} = b_j && \forall n_j \in N \\ &x_{ij} \leq \kappa_{ij} && \forall a_{ij} \in A \\ &p_{min_j} \leq p_j \leq p_{max_j} && \forall n_j \in N \end{aligned} \tag{3.14}$$

Alternatively, if π_k is defined as a *conforming simple path*[5] out of total Π directed paths between any pair of source-destination pairs, and x^{π_k} as a *path flow*[6] on it, then the collection of $\mathbf{x} = \{x^{\pi_k} \mid \pi_k \in \Pi\}$ represents the *network flow in path form*. The whole network flow can be represented as collection of conforming simple paths. The flows in individual arcs are (Ahuja et al., 1993):

$$x_{ij} = \sum_{\pi_k} \delta_{ij}^{\pi_k} x^{\pi_k} \tag{3.15}$$

where $\delta_{ij}^{\pi_k} = 1$ if a_{ij} is on path x^{π_k} and 0 otherwise.

[4] of a vector field is the rate at which "density" exits a given region of space and in the absence of the creation or destruction of matter, the density within a region of space can change only by having it flow into or out of the region (Weisstein, 1999b)

[5] directed path from source n_s to destination n_d node whose path flow is equal some quantity a, where $x_{ij} = a$ for forward arcs and $x_{ij} = -a$ for backward arcs

[6] amount of flow that is send from source node n_s to destination node n_d along path π_k

The formulation of the Minimum Cost Network Flow optimization problem in *path form* states:

$$min \quad z = \sum_{\pi_k \in \Pi} (x^{\pi_k} \sum_{a_{ij} \in \pi_k} c_{ij}) \tag{3.16}$$

subject to:

$$\begin{aligned} &\sum_{\pi_k} \delta_{ij}^{\pi_k} x^{\pi_k} \leq \kappa_{ij} \qquad &\forall \pi_k \in \Pi \\ &p_{min_j} \leq p_j \leq p_{max_j} \quad &\forall n_j \in N \end{aligned} \tag{3.17}$$

Although *arc form* and *path form* are completely equivalent, the *path form* is adopted here because it: a) determines flows in a more applicable way since in water supply systems water is always sent from some source node to some destination node and b) reduces the number of unknown variables since it uses path flows instead of arc flows as unknown variables. In addition such path form formulation is particularly convenient for the iterative or algorithms that build one solution on another since it allows ease reallocation of flows among alternative paths[7] keeping the delivery at the destination nodes constant.

Furthermore, the Minimum Cost Network Flow problem formulated in the path form does not need for explicit consideration of continuity equation at intermediate nodes since the flow is always conserved on conforming paths. The sum of flows has to match external flow value only on source and demand nodes. The satisfaction of this constraint and the arc capacity constraint together, is often referred as the *feasibility* of the solution:

$$\begin{aligned} &\sum_{n_j : a_{ij} \in A} x_{ij} - \sum_{n_j : a_{ji} \in A} x_{ji} = b_j \quad &\forall n_j = n_s, n_d \\ &\sum_{\pi_k} \delta_{ij}^{\pi_k} x^{\pi_k} \leq \kappa_{ij} &\forall \pi_k \in \Pi \end{aligned} \tag{3.18}$$

In water supply system analysis, the satisfaction of the pressure constraint (equation 3.17) in addition to the previous constraints is called the *satisfiability* of the solution:

$$\begin{aligned} &\sum_{n_j : a_{ij} \in A} x_{ij} - \sum_{n_j : a_{ji} \in A} x_{ji} = b_j \quad &\forall n_j = n_s, n_d \\ &\sum_{\pi_k} \delta_{ij}^{\pi_k} x^{\pi_k} \leq \kappa_{ij} &\forall \pi_k \in \Pi \\ &p_{min_j} \leq p_j \leq p_{max_j} &\forall n_j \in N \end{aligned} \tag{3.19}$$

3.2. Method for the Integration of Environmental and Socioeconomic Aspects

From scenic beauty and recreational opportunities, through input into production processes to the necessary drinking, health and sanitation medium, water provides a complex set of values to individuals and benefits to the society, so called *use values*. At the same time, water is inherent element of the environment and provide a variety of values for further development of life on earth, so called *non-use values*. Beside beneficial, sometimes referred as positive, water use values such as the contribution to better health and living standard or better living

[7]the ones that connect the same source-destination combination but have different set of arcs

conditions for plants and animals, some damaging, or negative, water use values, such as flooding, population migration, conflicts over water uses or decrease of biodiversity, can be identified. As explained in subchapter 2.2 on page 12 in order to identify and quantify these water use values as a consequence of some project or actions the functional dependencies between some parameters of the systems and their environmental and socioeconomic impacts will be used.

3.2.1. Representation of Water Supply System's Impacts

In the previously stated general formulation of the Minimum Cost Network Flow problem the term **cost** stands for the negative impacts that a water supply system may cause in economic, environmental, social and quality of a services domain. Since the main aim of man-made projects or actions is not the costs minimization but instead "the maximization of benefits keeping in mind cost considerations" (Walski et al., 2003), it is necessary to redefine this term in order to include positive consequence (**benefits**). The simplest way to achieve this is to use **net-costs** (c) defined as difference between negative and positive costs or impacts:

$$c(x) = costs(x) - benefits(x) \tag{3.20}$$

As discussed in the previous chapter, the identification and quantification of costs and benefits of an engineering project from an economic, environmental, social and system's quality point of view is not an easy task and relay on approximation methods. Examples that have been already applied in the analysis of water systems include statistical procedures (O'Neill, 1972; Roy et al., 1992), use of Satisfaction or Performance Indexes (de Neufville, 1970; Hellström et al., 2000; Seager, 2001; Foxon et al., 2002), use of Environmental Impact Assessment (Munn, 2006; Gunnerson, 1977; Petts, 1999), Strategic Environmental Assessment (Thérivel and Partidário, 1996), Life Cycle Assessment (Curran, 1996; ISO, 1997; Tillman et al., 1999) or Material Flow Analysis (Bringezu et al., 1997) method and modelling procedures for the assessment of economic costs (Lindsey and Walski, 1982; Clark et al., 2002) or for assessment of environmental impacts (Chadwick, 2002; Finnveden and Moberg, 2005). The collective aim of all these methods is to identify some kind of functional relationship among system properties and their direct or indirect impacts. Since the main focus of the presented study are not the individual methods for the evaluation of the impacts of water supply systems, but rather the development of the general framework that allows for an integrated analysis of these impacts, it is assumed that the individual impacts (costs and benefits) can be expressed as single-variable function of some system parameter. The typical forms of the parameter-impact relationships, or net-cost functions are presented in Figure 3.2.

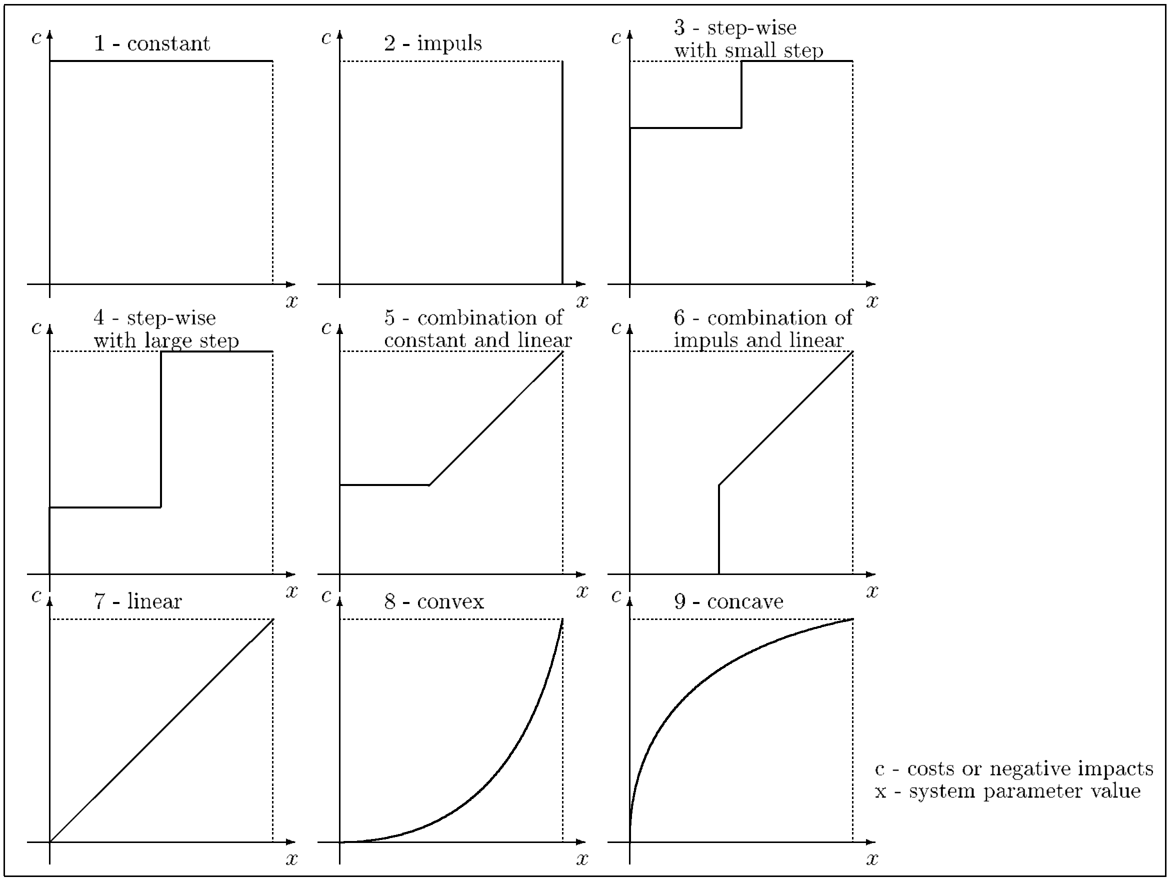


Figure 3.2.: Adopted typical forms of cost (negative impact) functions

These functional dependencies have to be representative for most of the impacts of water supply systems. For example, constant impacts or the ones that appear only when some value of some system parameter is reached such as building costs or benefits of water provision may take forms presented in graph 1 and 2 in Figure 3.2. The investment costs in construction of new system components or upgrade of existing ones are mainly approximated with unit cost coefficients that basically stand for different combination of constant and linear functional dependencies such as in graphs 5, 6 and 7 in Figure 3.2. Furthermore, complex dependencies such as the decrease of marginal cost with the scale of the system, so called "economy of scale", are often substituted with some kind of concave form functions as in graph 9 in Figure 3.2. The environmental impacts as consequence of human activities are often approximated as very small for small size actions with an exponential increasing trend with the increase of the size of the actions. A typical examples would be a decrease in groundwater level with the increase of the well water withdrawal, a decrease in river flow fluctuations with size of river intake or river impoundments or the reduction in water species number and variability with the reduction of wetland area. Such concave functions are presented in graph 8 in Figure 3.2. Finally, social impacts of the provision of water are often achieved incrementally and step-wise functions as in graphs 3 and 4 in Figure 3.2 are often very suitable for their mathematical formulation.

The above presented functions have to be accommodated for each individual impact. The parameters of the functions have to be adjusted to reflect the actual impact of some water supply system parameter. Since the adopted functions can be very easily mathematically formulated (Equation 3.21) with two equations and only three parameters p, q, r, the accommodation of each individual functional dependency (e.g. investment costs, groundwater level, wetland function, etc.) to some system parameter (e.g. pipe diameter, intake capacity, withdrawal value, etc.) is easily achievable.

$$c(x) = \begin{cases} r & if \quad x \leq p \\ (q-r)/p & if \quad x \geq p \end{cases} \qquad and \qquad c(x) = px^q + (1-p)x^r \tag{3.21}$$

3.2.2. Scaling of Impact Functions

Although it is possible to select one system parameter, usually flow x, as an independent variable for all parameter-impact functions $c^l(x)$, different water supply systems impacts will be expressed in different value units such as money, water level decrease, user satisfaction, etc.. If these are to be compared and summed they have to be brought to the comparable scale. As suggested by Haith and Loucks (1976) a simple way to enable comparison among such different sort of values is to use *scaling functions* $s(c)$ to transform all functions into dimensionless functions $C(x) = s(c(x))$ often called **unit-functions**. All unit-functions have values in the same range, for example $[0, .., 1]$, where 0 stands for minimum and 1 for maximal impacts. The transformation process itself is presented in Figure 3.3.

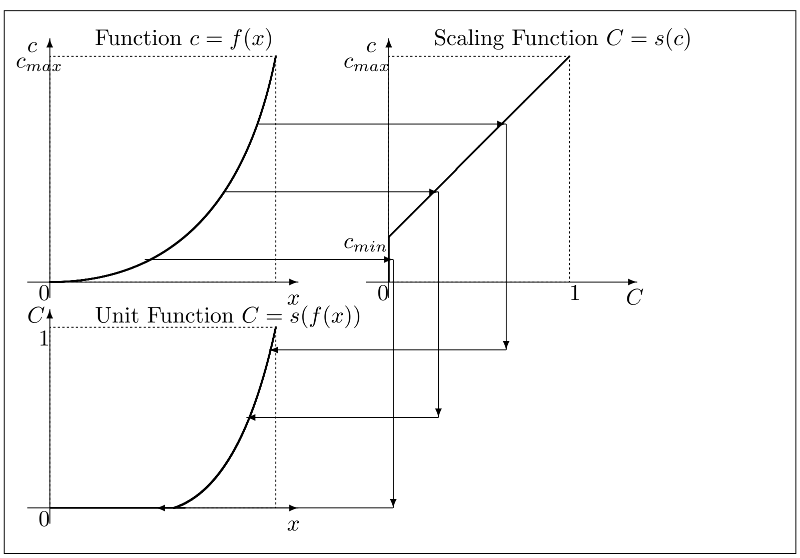


Figure 3.3.: Transformation of a function to the unit-function

Scaling functions determine the range of interest (minimum and maximum values) and the form of value transformation. Although scaling functions may take various forms, the linear form is the most often used one. Its mathematical representation is:

$$C(x) = s(c(x)) = \frac{c(x)}{c_{max} - c_{min}} + c_{min} \tag{3.22}$$

The Minimum Cost Network Flow optimization problem is then formulated as:

$$\begin{aligned} &min \quad z = \sum_{a_{ij} \in A} C_{ij}(x_{ij}) \qquad \text{or alternatively} \\ &min \quad z = \sum_{\pi_k \in \Pi} \sum_{a_{ij} \in \pi_k} C_{ij}(x^{\pi_k}) \end{aligned} \tag{3.23}$$

subject to:

$$\begin{aligned} &x_{ij} \leq \kappa_{ij} && \forall a_{ij} \in A \\ &p_{min_j} \leq p_j \leq p_{max_j} && \forall n_j \in N \\ &\sum_{n_j : a_{ij} \in A} x_{ij} - \sum_{n_j : a_{ji} \in A} x_{ji} = b_j && \forall n_j \in N \quad \text{or alternatively} \\ &\sum_{\pi_k} \delta_{ij}^{\pi_k} x^{\pi_k} \leq \kappa_{ij} && \forall \pi_k \in \Pi \end{aligned} \tag{3.24}$$

Having already stated that the impacts of a water supply system have to be evaluated from the economic, environmental, social and quality of a services point of view, it is imposed that the total impact is composed of:

$$C(x) = C^{ecn}(x) \quad + \quad C^{env}(x) \quad + \quad C^{soc}(x) \quad + \quad C^{syst}(x) \tag{3.25}$$

where $C^{ecn}(x)$, $C^{env}(x)$, $C^{soc}(x)$ and $C^{syst}(x)$ stand for economic, environmental, social and system quality impacts for some value of system parameter x, respectively. Having brought the impacts to the same scale is just a necessary prerequisite for their integration. In order to make comparison among different impacts or to find total impacts of some system, the relative utility (importance) of individual impacts to the decision makers have to be considered.

3.2.3. Multiple Criteria Analysis of Impact Functions

Management of water supply systems involves making choices among technically feasible alternatives, where these choices are often governed by economical and financial aspects, social acceptability or possible environmental impacts of the foreseen projects or actions (Linsley et al., 1992). In addition, each project alternative or different set of actions provide different qualitative performance of the system. In order to compare among these different aspects the **utility** (worth, value, convenience, importance) of each of them is the most important term that has to be defined. There are two approaches: 1) the economic definition of *utility* (social welfare) denoted as the capacity to satisfy human desires, usually measured by the price someone is willing to pay or willing to accept monetary compensation for gains or losses of some value (Johansson, 1993), and 2) the Decision Theory definition of *utility* denoted as a measure of the desirability of consequence of courses of action with which the decision maker chooses the alternative depending on his individual preferences and risk acceptability (Krippendorff, 2002). Since the first approach assumes that a social welfare function is a sum of similar utility functions of individuals in the society and demands for the quantification of all criteria in monetary terms it is often criticized for the types of analysis where individually governed criteria (e.g. environmental, quality of services) are to be considered (Pearce and Markandya, 1993). In contrast, the Decision Theory definition of utility establishes preferences between options by referencing to an explicit set of objectives that the decision making body has identified, and for which it has established measurable criteria to assess the extent to which the objectives have been achieved (DTLR, 2001). For the integrative consideration of economic, environmental, social and system quality objectives in the analysis of water supply systems second approach is adopted.

A demonstration of importance of different criteria in water supply systems is given by Munasinghe (1997). As illustrated in Figure 3.4 the author suggest that for many existing water supply systems at first may be possible to identify some actions that lead to simultaneous improvement over all criteria (economic, social, and environmental). After reaching this "win-win" scenario, further improvements on one criteria are possible only by decreasing one another, so called trade-off. The systematic approach that help to control these trade-offs among different objectives according to the preferences of decision makers is referred as **Multiple Criteria Decision Making** or **Multiple Criteria Decision Analysis** (MCDM or MCDA)[8].

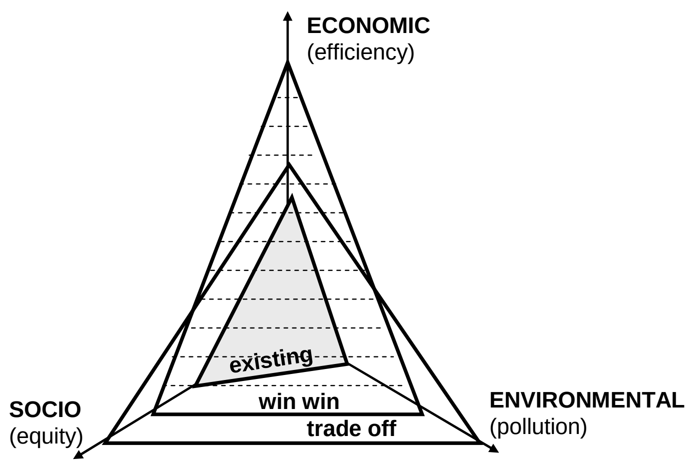


Figure 3.4.: Multi criteria analysis of water supply systems [source: Munasinghe, 1997]

Recognition of multiple and conflicting objectives and criteria in many disciplines, significantly advanced the development of MCDA in the last decades. The MCDA provide for possibility to quantify the changes in the solutions depending on the changes in the utilities toward different objectives and is based on the idea of Pareto-optimal solution, "one for which no other solution exists that will yield an improvement in one objective without causing degradation in at least one other objective" (Cohon, 1978).

Mathematically a point $p^* \in P$ is defined as being *Pareto-optimal* (Pareto, 1896), *non-dominated* (Kuhn and Tucker, 1951), *non-inferior* (Cohon, 1978) or *satisficing* (Zeleny, 1982) if and only if there exists no other point $p \in P$ such that:

$$\begin{array}{lll} 1.) & z^i(p) \leq z^i(p^*) & \forall i \in \{1...L\} \\ 2.) & \exists \mathrm{j} & z^j(p) < z^j(p^*) \end{array} \tag{3.26}$$

[8] the discipline aims at supporting decision maker(s) that deal with conflicting objectives whose foundations are in the mathematical theory of optimization under multiple objectives (Ehrgott and Gandibleux, 2003)

where L is a set of all objectives, P set of all feasible solutions and z^i is the evaluation of the function on $i-th$ objective. If different objectives are in conflict, the Pareto-optimal solutions form a, so called *Pareto-front* (*efficient frontier*) that, for the case of two objective function z_1 and z_2, may look like the one in Figure 3.5.

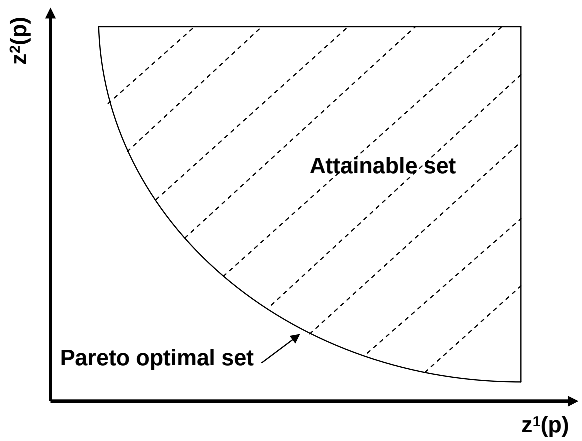


Figure 3.5.: Pareto-optimal set, [source: Liu et al., 2001]

Which solution form a Pareto-optimal set is going to be selected as globally optimal, depends on the decision makers' utilities toward objectives, z^1 and z^2. Since the assessment of the preferences and utility functions of decision makers is a very difficult and complex process the identification of the Pareto-optimal Set provides for a possibility to avoid explicit definition of these utilities and for the selection of the optimal solution based on trade-off among identified optimal alternatives. Haith and Loucks (1976) suggests that "instead of trying to derive utility functions of decision makers, the analysts has to concentrate on delineating the possible trade-off between various objectives by defining the alternatives and evaluating each alternative based on criteria expressed in, for decision makers, meaningful terms". In other words, this means the identification of the Pareto-optimal set and restricting of the decision making process to the set of optimal alternatives.

In most general form the optimization problem for multiple criteria can be stated as (Zeleny, 1982):

$$\begin{array}{lll} sat. & z = \{z^l, \forall l = 1...L\} & or \\ min. & z = \{z^l_w, \forall l = 1...L, \forall w = 1...W\} & \end{array} \tag{3.27}$$

where $sat.$ stands for satisficing solution, L is set of considered criteria, W is set of combinations of the decision makers utilities toward different criteria and z^l_w is the evaluations of the suggested alternatives according to the criteria l for a decision makers utility combination toward different criteria w.

Accordingly the Minimum Cost Network Flow problem for consideration of economic, environmental, social and systems quality criteria in its arc and path flow can be now rewritten as:

$$\begin{aligned} &sat. \quad z = \{z^l, \forall l = ecn, env, soc, qual\}\ , \quad z^l = \sum_{a_{ij} \in A} C^l_{ij}(x_{ij}) \qquad\qquad or \\ &sat. \quad z = \{z^l, \forall l = ecn, env, soc, qual\}\ , \quad z^l = \sum_{\pi_k \in \Pi} \sum_{a_{ij} \in \pi_k} C^l_{ij}(x^{\pi_k}) \end{aligned} \tag{3.28}$$

subject to:

$$\begin{aligned} & x_{ij} \leq \kappa_{ij} && \forall a_{ij} \in A \\ & p_{min_j} \leq p_j \leq p_{max_j} && \forall n_j \in N \\ & \sum_{n_j : a_{ij} \in A} x_{ij} - \sum_{n_j : a_{ji} \in A} x_{ji} = b_j && \forall n_j \in N \quad or\ alternatively \\ & \sum_{\pi^k} \delta^{\pi_k}_{ij} x^{\pi_k} \leq \kappa_{ij} && \forall \pi_k \in \Pi \end{aligned} \tag{3.29}$$

3.2.4. Integrative Analysis of Fixed and Variable Impacts

An additional characteristics of water supply systems, that has to be considered during the definition of the optimization problem, is the fact that their impacts are principally divided into the ones that occur during construction (**fixed**) and the ones that occur during operation and use of the facilities (**variable**). Since any of these two may be of prevailing influence, they have to be simultaneously considered. Furthermore, the fixed impacts are only applicable to the potential (not yet existing) elements or the ones that are considered for expansion or rehabilitation, and an additional integer variable y_{ij} had to be introduced in order account for this distinction. This enables the calculation of the total impacts as the sum of fixed and variable ones (Figure 3.6):

$$C_{ij} = C_{fix_{ij}} y_{ij} + C_{var_{ij}}; \quad y_{ij} = 0 \vee 1 \tag{3.30}$$

where y_{ij} takes 0 for existing elements and 1 for potential or elements for rehabilitation.

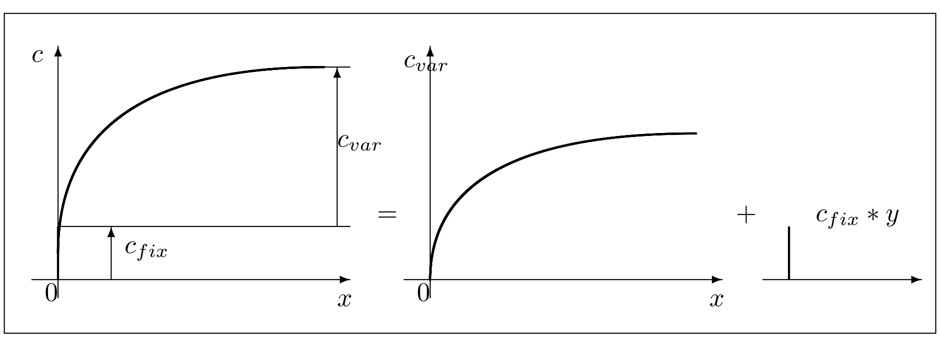


Figure 3.6.: Integration of fixed and variable costs (impacts)

The combining of fixed investment costs, degradation of the environment, changes in river regime due to impoundments, etc. with operation costs, groundwater level reduction, reduction of river flows, etc.) provides for an integrative analysis of existing systems, their expansion or rehabilitation and the building of new ones. But in order to be comparable fixed and variable impacts have to be brought to the same time horizon. For example economic consequence of water supply are assessed in months or years while some environmental impacts may last for thousands of years.

The economic values are brought to the same time scale mainly by using the Time Value of Money concept. It is based on the premise that most people prefer to receive money today, rather than the same amount in the future. The difference in the value of money today (PV) and in some future time (FV) is caused by opportunity cots (i.e. loss of value since money is not put to productive use) and risk over time (e.g. risk of inflation). In Time Value of Money calculations these two are expressed with the interest rate (r). For some time period n with the constants interest rate r the present (PV) and the future value of money (FV) can be equated using following formulas (Copeland et al., 1998):

$$PV = \frac{FV}{(1+r)^n} \quad and \quad FV = PV(1+r)^n \tag{3.31}$$

Since the future benefits and costs are usually not a single value but rather a stream of values (e.g. credit payments, operation costs, etc.) the time value of money are usually expressed to their annuity (A) (Copeland et al., 1998):

$$PVA = A\frac{1-\frac{1}{(1+r)^n}}{r} \quad and \quad FVA = A\frac{(1+r)^n-1}{r} \tag{3.32}$$

where PVA-present value to an annuity, FVA-future value to an annuity and A-is the annuity or the individual value in each compounding[9] period.

Based on this it is possible to discount the value of a projects, company or anything else for which some nominal future value (FV) can be defined to the appropriate present value (DPV) simply by summing its successive present values in compounding periods t (Copeland et al., 1998):

$$DPV = \sum_{t=n}^{t=0} \frac{FV_t}{(1+r)^t} \tag{3.33}$$

Adapting a multi-objective approach, Loucks and Gladwell (1999) suggested to use the weighted sum of successive future present values in order to encompass the different value of money in different time periods.

$$DPV = \sum_{t=n}^{t=0} a_t \frac{FV_t}{(1+r)^t} \tag{3.34}$$

where a_t is a weight of present values of in period t and $\sum a_t = 1; t \in 1,..n$.

Such a formulation of discounted present value of some future costs is very flexible in terms of setting the preferences toward future benefits and costs and can be easily adopted for the discounting of environmental and social impacts too. Only, the interest rate (r), or discount rate, has to be accommodated to encompass the opportunity and risk of environmental and social aspects. Unfortunately this can not be done with the certainty. But looking at the current trends and preferences of our society it may be assumed that the environmental

[9]length of time in which an asset can generate cost or benefits

and social aspects will obtain ever greater importance. To some degree this can be then represented with greater interest rates.

For the introduced notation the discounting of the variable impacts can be written as:

$$DC_{var} = \sum_{t=n}^{t=0} a_t \frac{C_{var}}{(1+r)^t} \tag{3.35}$$

where DC_{var}-are discounted variable impacts to the present value, r-interest(discount) rate, n total number of time periods t, C_{var}-variable impacts in future time periods and a_t their corresponding weights.

Together with the introduced y_{ij} variable, the Minimum Cost Network Flow problem in its arc and path form can be now stated as:

$$\begin{aligned}
&sat. \quad z = \{z^l, \forall l = ecn, env, soc, qual\}, \quad z^l = \sum_{a_{ij} \in A} (DC^l_{var_{ij}}(x_{ij}) + C^l_{fix_{ij}}(x_{ij}) y_{ij}) \qquad or \\
&sat. \quad z = \{z^l, \forall l = ecn, env, soc, qual\}\ , \quad z^l = \sum_{\pi_k \in \Pi} \sum_{a_{ij} \in \pi_k} (DC^l_{var_{ij}}(x^{\pi_k}) + C^l_{fix_{ij}}(x^{\pi_k}) y_{ij})
\end{aligned} \tag{3.36}$$

subject to:

$$\begin{array}{lll}
x_{ij} \leq \kappa_{ij} y_{ij} & \forall a_{ij} \in A & \\
p_{min_j} \leq p_j \leq p_{max_j} & \forall n_j \in N & \\
\sum\limits_{n_j : a_{ij} \in A} x_{ij} - \sum\limits_{n_j : a_{ji} \in A} x_{ji} = b_j & \forall n_j \in N & or\ alternatively \\
\sum\limits_{\pi_k} \delta^{\pi_k}_{ij} x^{\pi_k} \leq \kappa_{ij} y_{ij} & \forall \pi_k \in \Pi & \\
y_{ij} = 0 \vee 1 & \forall a_{ij} \in A &
\end{array} \tag{3.37}$$

At the end it is important to stress that the inclusion of the discrete variable y_{ij} renders defined optimisation problem as a multi-variable one. This means that beside the flows (x_{ij}), the configuration of a network (y_{ij}) is also variable and have to be optimised. Since the configuration of a network controls its flows, it is suggested to decompose the problem and use one algorithm to create, evaluate and identify optimal network configuration and another one to identify the minimum cost flow solution for each of these configurations.

3.3. Methods for the Solution of the Optimisation Problem

Numerous optimization techniques that have been successfully applied for the Minimum Cost Network Flow problem can be divided in three general categories (Bertsekas, 1998):

1. Primal cost improvement - iterative improvement of costs by constructing sequence of feasible flows,
2. Dual cost improvement - iterative improvement of dual costs by constructing sequence of prices[10].
3. Auction - generation of prices in a way that is reminiscent to real-life auctions but in addition to prices the algorithms iterate on flows, too[11].

In general they are all iterative procedures to obtain a solution of an optimization problem that satisfies the constraint conditions and the principal difference is the order in which the "closeness" to the optimum and the constraint conditions are satisfied (Jensen, 1980). Due to the fact that each of them is more convenient for a slightly different type of problem the prime criteria in the selection of optimization technique are the characteristics of the problem itself.

3.3.1. Characteristics of the Optimisation Problem

A prime characteristic that distinguishes the defined Minimum Cost Network Flow Problem from a standard one, "a least cost shipment of a commodity through a network in order to satisfy demands at certain nodes form available supplies at other nodes" (Bertsekas, 1998), is the implementation of linear, step-wise, convex and concave functions instead of cost coefficients. Most network algorithms are appropriate, or efficient, only for the linear cost functions because they select the direction of the search based on the gradients of the function at the point under the examination. The left graph in Figure 3.7 shows how convex functions can be approximated with many subsequent linear functions since the gradients of these new functions can be ordered in an increasing, or at least monotony non-decreasing, order.

[10] the original network problem, called primal, can be transformed to another problem, called dual, by transforming the constraints to the decision variables, called prices, and the decision variables to the constraints. The dual costs represent the difference between original costs and newly formed prices and if the original problem minimise costs than the dual problem maximise its dual costs (Bertsekas, 1998)

[11] the dependency between flows and prices and the termination of the algorithm is based on a property called *complementary slackness* that state that a solution is optimal if its primal and dual variables equal their primal and dual constraints at the same time (Minieka, 1978)

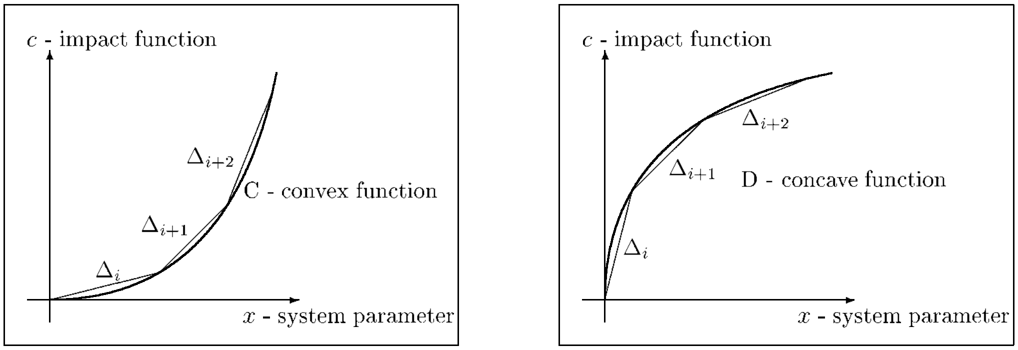


Figure 3.7.: Linear approximation of convex and concave functions

A similar approximation for the concave functions would lead a gradient oriented optimization procedure to select the upper segments of a function first, since they have lower gradients (right graph in Figure 3.7). Furthermore, the combination of different forms of cost functions, such as in Figure 3.2 on page 41, creates a discrete problem with numerous local optima[12] that are very hard to solve for global optimality[13]. In addition the discrete variable space constraints the use of the standard Linear Programming techniques and demands for some kind of numerical approximation in order to reduce the complexity of the problem (Vavasis, 1995). Due to the abundance of many similar real life problems, a large number of optimization techniques, so called Global Optimization Techniques, have been developed. Techniques that aim to generate solution for the discrete non-convex combinatorial problems can be generally divided into two categories Gray et al. (1997); Pintér (2005):

1. *Exact methods* - tend to guarantee the global optima but are constrained by problem formulation structure or high computational demands. They include Naive Approach, Enumerative Search, Parameter Continuation and Relaxation Methods, Branch and Bound and many others,

2. *Approximation methods* - are often computationally very efficient but inevitably contain a certain level of randomness within the search. Such methods do not guarantee a correct global solution but usually produce a very good ones. They include globalized extensions of Local Search, various Evolution Strategies, Simulated Annealing, Tabu Search, Approximate Convex Global Underestimation, Continuation methods and many others.

Since both groups have their advantages and disadvantages, in recent years there is a growing number of combinations of the methods from these two groups. A similar effort is made in this study and the Simulated Annealing method, as an robust, simple and efficient optimisation procedure, is embedded within the Branch and Bound algorithm, which advance exhaustiveness of the search and the identification of the global optima. Basically this means that the solution procedure is decomposed into the identification of the minimum cost flow solution for one network configuration (primal solution) achieved by Simulated Annealing,

[12] a solution optimal within a neighbouring set of solutions (Cook et al., 1997)
[13] the optimal solution of the whole solution space (Cook et al., 1997)

and the identification of the global optimal solution for all possible configurations (final solution) controlled by Branch and Bound algorithm. Since the proposed approach aims to iteratively improve the optimality of the solution its performances are significantly better if it starts from one pre-identified feasible solution (initial solution). Furthermore, by selecting a new iterative solutions only from a set of feasible ones, the computational performances of the procedure can be significantly improved. These basic optimisation steps as a part of the decision support in management of water supply systems are presented in Figure 3.8. A more detailed description follows.

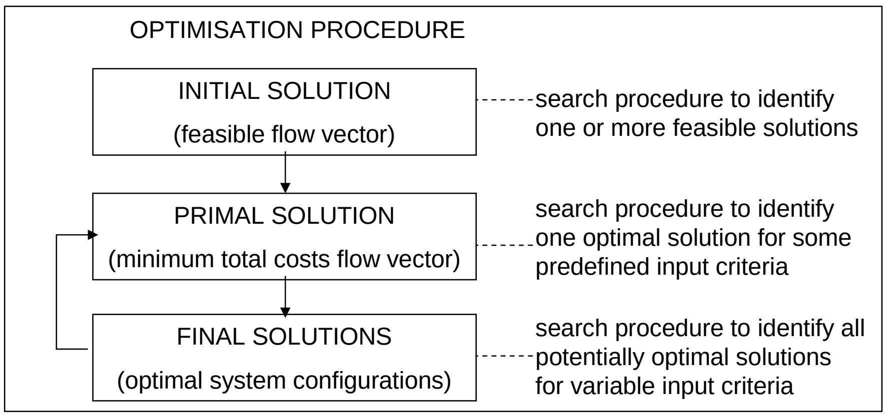


Figure 3.8.: Main steps of the optimisation procedure

3.3.2. Initial Solution with the Maximum Feasible Flow Method

A network flow solution that satisfies conservation constraints on nodes and arcs, but does not consider network costs, is called **initial or feasible solution**. Out of numerous network algorithms for the calculation of a such solution, the Maximum Feasible Flow Algorithm of Jensen (1980) has been selected mainly due to its simplicity. It is essentially based on the famous Ford Jr. and Fulkerson (1956) Min Cut-Max Flow theorem:

For any given network with capacities $\kappa_{ij} > 0$, the value of a maximal flow equals the value of a minimal cut,

where, a **cut** in a network $G_n = (N, A)$ is a partition (O, T) of N such that $O \subseteq N$, $\emptyset \neq O$, $T = \overline{O}$, in which $n_o \in O$ are origin and $n_t \in T$ terminal nodes in respective sets[14]. The arcs in cut are: $A^{O,T} = \{a_{ij} : n_i \in O, n_j \in T\}$ and the *capacity of cut* is $\kappa^{O,T} = \sum_{a_{ij} \in A^{O,T}} \kappa_{ij}$. The cut with the smallest capacity is called a *minimum cut*. In essence a minimal cut can be seen as a bottleneck in a network and the theorem states that the largest possible flow will equal the capacity of a bottleneck (Spelberg et al., 2000).

The Maximum Flow Feasible algorithm of Jensen (1980) starts with all flows equal zero and gradually increases flows on augmenting paths[15], for maximum possible flow augmentation,

[14]for the set of all nodes N the $\overline{O}$ is complement set to the set O in set N if it contains all elements of N that are not in O. (Weisstein, 1999a)

[15]network paths in which still some spare capacity (augmentation flow) exists.

till the bottleneck capacity is reached. Augmenting actually increases the network flows on forward and decreases them on backward arcs since the later ones have a negative flow value. The procedure is executed for all pair of nodes, one with unsatisfied positive and another with unsatisfied negative external flow until all external flows are satisfied or all path capacities are used to their maximum flow. At the end, if all external flows are not satisfied then a feasible solution of a problem does not exist. In order to deal with this an additional node, so called *slack node*, is introduced. This virtual node is with virtual arcs, *slack arcs*, connected with all source and demand nodes in order to accept the surplus and provide for the deficient external flows. The flow is routed to slack node only when all other node pairs are exhausted and total flow in it serves as the indication of the feasibility of a solution. Basic steps of the Maximum Feasible Flow Method are the following (Jensen, 1980):

1. *Initialize* - Set all arc flows to null $x_{ij} = 0$, $\forall x_{ij} \in A$, create slack node n_S and slack arcs form every source node to slack node a_{sS}, $\forall n_s \in N$ and from slack node to every destination node a_{Sd}, $\forall n_d \in N$.

2. *Maximum flow* - Find a node pair (n_s, n_d) with positive external flow on source and negative external flow on destination node and with still unsatisfied external flows, establish an augmenting path π_a among them and augment maximum flow amount possible $x^{\pi_a} = Min(\mid b_s \mid, \mid b_d \mid, Max(\kappa_{ij}, \forall a_{ij} \in \pi_a))$. Reduce the magnitude of the unsatisfied external flows at source and destination nodes for the augmented flow amount $\mid b_s \mid = \mid b_s \mid - x^{\pi_a}$, $\mid b_d \mid = \mid b_d \mid - x^{\pi_a}$.

3. *Control* - If external flow is not satisfied on either source $b_s > 0$ or demand node $b_d < 0$, search for another complementary node (demand for source node and source for demand node) and repeat the step two for this new node pair. Since the algorithm does not leave a node before it satisfies its external flow, after "visiting" every source and destination node for at least once the algorithm should find a feasible solution. If all nodes have been already examined and the external flow at some node is still not satisfied, the algorithm establish a path to the slack node and allocate unsatisfied flow to this path.

As previously stated the total water flow at the slack node b_S is an indicator of the feasibility of the solution. It is equal the sum of flows on all slack arcs and is calculated by the following formula:

$$b_S = \sum_{n_j : a_{Sj} \in A} x_{Sj} - \sum_{n_j : a_{jS} \in A} x_{jS} \quad \forall n_j = n_s, n_d \tag{3.38}$$

$b_S = 0$ shows than the total supply and demand external flows are equal and the feasible solution on the network has been found. $b_S > 0$ shows the existence of surplus supply for the found feasible solution and $b_S < 0$ the existence of demands which can not be satisfied due to not enough supplies or capacities on a network. Since the usual approach for identification of the network optimal solution is to start with the network configuration with the maximum potential element's number and sizes and then try to gradually reduce the costs by reducing element's sizes or taking some elements out of the network, if there is no feasible solution for the first configuration there will be no feasible solutions for all others too. This has to be corrected, either by adding new potential elements or by increasing the set of element's potential sizes, before further proceeding in the optimization procedure.

3.3.3. Primal Solution with the Simulated Annealing Method

The procedure for the identification of a minimum cost flow solution for one system configuration, referred as the **primal solution**, has to deal with a discrete network problem defined on linear, step-wise, convex and concave cost functions. Even more, the procedure has to be robust enough to handle many instances of local optima, many different constraints (e.g. capacity of arcs, continuity on nodes, pressures in network, etc.), different initial conditions (existing and new systems) and to allow accommodation for different types of optimization problems (i.e. planning, design, operation problem).

Among various heuristic procedures the Simulated Annealing (Kirkpatrick et al., 1983; Cerny, 1985) is selected mainly due to its conceptual simplicity and proved robustness. Similarly as heat induce atoms of crystals to wander randomly through the states of higher energy until they find a state with the lower one, Simulated Annealing uses Metropolis-Hastings algorithm (Metropolis et al., 1953; Hastings, 1970) to ultimately move to the better point (one with lower energy state) and probabilistically evaluate the possibility to accept the worse point (one with higher energy state) too. The used probability is described with the Maxwell-Boltzmann distribution that imitates the exponential reduction of the energy variations (corresponds to the acceptance of the worse point) with the reduction of the temperature of gases. This allows Simulated Annealing to go uphill and downhill at the beginning in very large steps and then by reducing the probability of accepting the uphill move to focus on finding an optimal solution. Beside the temperature change, or so called *cooling-schedule* that must allow the algorithm to make enough uphill and downhill moves in order to identify global optima, second critical parameter is the *neighbourhood function* or the way of creating of new random points. This function is application specific and, in order to achieve the effective use of the method, it has to be accommodated in a way that the difference between old and new points is in the same order of magnitude as the probability of acceptance of the worse points. This is where it is possible to use the advantage of the graph representation of water supply systems and instead of random creation of new points (flows on individual arcs), use the simple conforming paths to effectively create new feasible solutions by exchanging flow on alternative paths. In addition, for each randomly created flow change the selection of the alternative augmenting paths can be improved by identifying the current minimum cost ones and their prioritising. Keeping in mind that the identification of the optimal solution is computationally very demanding (demands simultaneous examination of all source-demand node combinations), the introduced improvement in the direction of the search and the constraint to the feasible range only, significantly advance the total efficiency of the algorithm. Moreover, the independence from the initial solution and the convergence of the algorithm, are additionally improved by extending the Simulated Annealing to simultaneously iterate on a set of solutions $\mathbf{X} = (\mathbf{x_1}, \mathbf{x_2}, ..., \mathbf{x_N})$ instead on working on only one solution. The main steps of the used Simulated Annealing method are the following:

1. *Initialize* - Starting from one feasible solution, create a set of N initial solutions $\mathbf{X}'$ by randomly exchanging the flow on conforming simple paths x^{π} for all source-demand nodes combinations Π. These set of randomly created solutions and the set of their function values Z', represent the starting points for the rest of the algorithm. Set

the Simulated Annealing schedule parameters: T_{max} - initial temperature, ΔT - temperature decrease parameter, T_{min} - lowest temperature, N_{max} - maximal number of changes at each temperature and N_{suc} - maximal number of successful changes at each temperature $\dot{T}$[16].

2. *Change* - For every solution $\mathbf{x}'$ from a set X' invoke a random flow change Δx_i on all network paths x^{π_i}, $\forall x^{\pi_i} \in \Pi$, identify the set of all augmenting paths that can compensate this change Π^A and find the one x^{π_j} with the minimum cost path flow change $\Delta z_j^{\pi} = Min(z_k^{\pi}(x^{\pi_k} + \Delta x) - z_k^{\pi}(x^{\pi_k})), \forall x^{\pi_k} \in \Pi^A$. By reallocation the flows Δx_i from the paths x^{π_i} to their minimum cost augmenting paths x^{π_j} for all source-destination combinations $\forall x^{\pi_i} \in \Pi$, a new solution $\mathbf{x}''$ is created. Its function value z'' and the difference from the previous solution $\Delta z = z'' - z'$ are calculated and the acceptance is evaluated according to the following probability (Metropolis et al., 1953):

$$P = \begin{cases} 1 & \text{if} \quad \Delta z \leq 0 \\ e^{-\Delta z}/B\dot{T} & \text{if} \quad \Delta z > 0 \end{cases} \tag{3.39}$$

where $\dot{T}$ is the temperature at the current energy level $\dot{}$ and B is constant that relates temperature to the function value (similar to Boltzmann's constant for temperature and energy). If $\Delta z < 0$ the probability P is greater then 1 and the method accept this change, while for $\Delta z > 0$ probability depends on the current temperature of the algorithm. Since the temperature reduces with each new energy level of the algorithm (cooling schedule), the above stated probability also reduces with the progressing of the algorithm. At each temperature the creation of the new solutions x'' and evaluation of their acceptance is repeated until the maximal number of changes (N_{max}) or maximal number of successful changes (N_{suc}) is reached.

3. *Evaluate* - The newly created set of solutions $\mathbf{X}'' = (\mathbf{x_1''}, \mathbf{x_2''}, ..., \mathbf{x_N''})$ with its function values set $Z'' = (z_1'', z_2'', ..., z_N'')$ is sorted in an increasing order list $Z'' = [z_1'', z_2'', ..., z_N'']$ and if minimum temperature is reached $\dot{T} \leq T_{min}$ the solution at the first place is the optimal one. If the temperature is greater than the minimum one $\dot{T} \geq T_{min}$ it is decreased according to the cooling schedule (geometric temperature decrease $\ddot{T} = \dot{T} * \Delta T$ is adopted) and steps 2. and 3. are repeated.

As proved by theoretical studies of Gidas (1985), the Simulated Annealing procedure converges to an optimal solution if and only if the control parameter (temperature T) is decreased according to the following function:

$$\ddot{T} = Q/log(\dot{T}) \tag{3.40}$$

where $\ddot{T}$ and $\dot{T}$ are the temperature values at the consecutive energy levels and Q is a constant term depending on the depth of local minimum generated by the transformation used to pass form one solution to another. Since, the depth of a local minima is hard to assess in advance and the temperature decrease according to the above formula requires exponential number

[16] the temperature corresponds to the energy level and will be noted with the indices $\dot{}$ while the iterations at one temperature will be noted with the $'$ indices

of iterations, in many practical implementations of the Simulated Annealing, a geometric temperature decrease is used. Therefore the initial temperature has to be large enough to avoid poor quality local optima and the temperature decrease and total number of allowed iterations must be tuned, mainly by trial and error, so that the algorithm reach the global optima with desired accuracy. These restriction of the above proposed method has to be kept in mind for its later application.

In addition, to the satisfaction of the feasibility constraint each identified solution has to satisfy the satisfiability constraint too (equation 3.19 on page 39). Satisfiability provides for the satisfaction of the second most important parameter in water supply system, namely pressure distribution, and may even have larger importance for the selection of an optimal solution than the costs itself. Since the pressures are not independent variables (depend on flows, system capacities, topographic characteristics and operation of pressure control devices) they are not introduced as new decision variables but instead the satisfaction of their minimum and maximum values is considered through penalties or artificial increases in the total costs for damage of the pressure constraints:

$$z = \begin{cases} z + \sum\limits_{n_j \in N} (p_{min_j} - p_j)\Delta P & if \quad p_j < p_{min_j} \\ z & if \quad p_{min_j} < p_j < p_{max_j} \\ z + \sum\limits_{n_j \in N} (p_j - p_{max_j})\Delta P & if \quad p_j > p_{max_j} \end{cases} \tag{3.41}$$

where z is the function value, p_j, p_{max_j} *and* p_{min_j} are calculated, maximal and minimal pressure at some node n_j and ΔP is a penalty constant. It is important to notice that the above defined penalty test does not reject the solutions that do not satisfy the satisfiability criteria but only add upon their costs, which allows that the solutions with very low cost value but small deviation from satisfiability (performance) criteria also come into the final solution set. These solutions may be of the great importance for the risk-oriented design of water supply systems.

3.3.4. Adaptation of the Simulated Annealing for Multi-objective Problem

As just described the iterative search for the optimal solution of the Simulated Annealing algorithm is based on the evaluation of the differences among the new and the previous solution $\Delta z = z'' - z'$ where $\Delta z > 0$ corresponds to the improving and $\Delta z < 0$ to the deterioration of the single objective (criteria) function value z. But the alternatives formulated by water resource managers generally attempt, explicitly or implicitly, to achieve qualitative integration of numerous economic, political, social and technological objectives defined through different criteria z^l (Haith and Loucks, 1976). For such a multi-criteria optimization problem it is not so easy to define the overall function value because it is an aggregate of function values on different criteria. Especially when the improvement on one criteria causes degradation on another is hard to be evaluated for the overall performance. For the case of a two criteria problem three possible cases of mutual improvements or degradation on individual criteria are presented in Figure 3.9.

- *case a*: $\Delta z^l \leq 0$, $\forall l \in L$ (improvement on all criteria)
- *case b*: $\exists l_1, l_2$, $\Delta z^{l_1} < 0$ and $\Delta z^{l_2} > 0$ (simultaneous improvement and deterioration)
- *case c*: $\Delta z^l \geq 0$, $\forall l \in L$ (deterioration on all criteria)

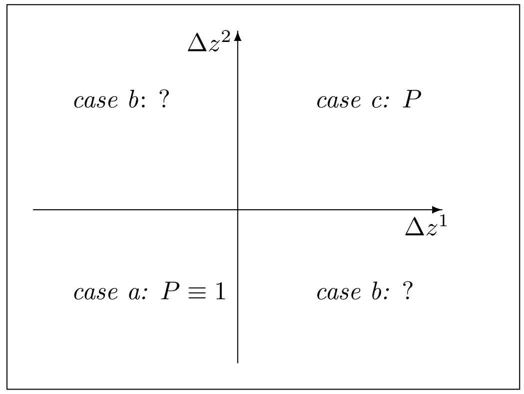


Figure 3.9.: Acceptance problem in multi-criteria optimization [source Ulungu et al., 1999]

In order to identify the Pareto-optimal[17] solutions for such a multi-criteria problem and enable the treatment of the simultaneous improvement and deterioration on different criteria with single-criteria optimization algorithm, such as the Simulated Annealing, Ulungu et al. (1995) developed a so called Multi-Objective Simulated Annealing (MOSA) method. In order to scale the multidimensional criteria space into a mono-dimensional one where the classical Simulated Annealing decision rule holds, the MOSA method introduces a *criterion scaling function.* Its purpose is to allocate utilities to the different criteria in order to enable their summing up. Although many different forms of criterion scaling functions may be used, the authors prove that, due to the stochastic nature of the algorithm, caused difference are very small and recommend the the simplest of all to be used. This is, so called, *weighted sum* and is mathematically expressed as:

$$z_w(z, w) = \sum_{l=1}^{L} w^l z^l, \quad \sum_{l=1}^{L} w^l = 1, \quad w^l \geq 0 \quad \forall l \in L \tag{3.42}$$

where w^l and z^l are the weight and function value for the criteria l from a set of total criteria L.

[17] "one for which no other solution exists that will yield an improvement in one objective without causing degradation in at least one other objective" (Cohon, 1978)

With the criterion scaling function, the Multi-Objective Minimum Cost Network Flow problem, in its arc and path form, can be stated as:

$$\begin{array}{llr} min. \quad z = \{z_w, \forall w \in W\}, & z_w = \sum_{l \in L} (w^l \sum_{a_{ij} \in A} (DC^l_{var_{ij}}(x_{ij}) + C^l_{fix_{ij}}(x_{ij}) y_{ij})) & or \\ min. \quad z = \{z_w, \forall w \in W\}\ , & z_w = \sum_{l \in L} (w^l \sum_{\pi_k \in \Pi} \sum_{a_{ij} \in \pi_k} (DC^l_{var_{ij}}(x^{\pi_k}) + C^l_{fix_{ij}}(x^{\pi_k}) y_{ij})) & \end{array} \tag{3.43}$$

subject to:

$$\begin{array}{lll} x_{ij} \leq \kappa_{ij} y_{ij} & \forall a_{ij} \in A & \\ p_{min_j} \leq p_j \leq p_{max_j} & \forall n_j \in N & \\ \sum_{n_j : a_{ij} \in A} x_{ij} - \sum_{n_j : a_{ji} \in A} x_{ji} = b_j & \forall n_j \in N & \textit{or alternatively} \\ \sum_{\pi_k} \delta^{\pi_k}_{ij} x^{\pi_k} \leq \kappa_{ij} y_{ij} & \forall \pi_k \in \Pi & \\ y_{ij} = 0 \vee 1 & \forall a_{ij} \in A & \\ \sum_{l=1}^{L} w^l = 1, & w^l \geq 0 & \forall l \in L \end{array} \tag{3.44}$$

It is not only that such defined weights make different criteria commensurable but it is also that they provide for the possibility to develop different alternatives simply by varying the importance of the different criteria. Basically this provides for an easy way for the identification of the set of Pareto-optimal solutions, since each solution from this set corresponds to the one combination of the weights on criteria. As expected, Ulungu et al. (1999) proved that a selected set of weights induces a privileged search direction on the efficient frontier and limit a procedure to generate only a subset of potentially efficient solutions in that direction. In order to avoid this limitation in identification of the complete Pareto-optimal set, the authors suggest the generation of the wide diversified set of weights and re-run of the procedure for each weights combination. Basically the procedure does not need to be re-run for a very large number of weights combinations but only for the dominated (Pareto-optimal) combinations. In addition the integration of the criteria weights in the problem formulation, enables for a very ease identification of the single-criteria solutions (extreme solutions that lay at the borders of the solution space) simply by allocation maximal weight to only one criteria. These solutions can be very useful in the model validation phase since they present the effects of the single-criteria-oriented decisions.

The MOSA Algorithm consists of the following basic steps:

1. *Weights* - Generate a large set Ω of diversified weight combinations $W = (w^l)$ where each individual weight w^l has uniform distribution toward different criteria l: $w^l \in \{0, 1/r, 2/r, ..., (r-1)/r, 1\}$ and r is the discretisation factor. Out of this set, by using the pairwise comparison, the set of dominant (Pareto-optimal) combinations Ω^D is selected for further running of the algorithm. For each weight combination $W_i \in \Omega^D$ the following steps are then repeated.

2. *Initialize* - Set the Simulated Annealing schedule parameters: T_{max} - initial temperature, ΔT - temperature decrease scheme, T_{min} - lowest temperature, N_{max} - maximal number of changes at each temperature and N_{suc} - maximal number of successful changes at each temperature.

3. *Change and Evaluate* - In the first iteration create a set of random feasible solutions $\mathbf{X}' = (\mathbf{x}')$ and in all others use previously described Simulated Annealing method to produce a set of new solutions $\mathbf{X}'' = (\mathbf{x}'')$. For each of these solutions x'' its function values according to the each criteria $z'' = \{z''^1, z''^2, ..., z''^L\}$ are evaluated and changes on each individual criteria Δz^l are calculated. These are scaled (weighted) according to the current weights combination $z_w(z,w) = \sum_{l=1}^{L} w^l z^l$ and the aggregate function change is calculated as $\Delta\ s = z_w(z^l, w^l)'' - z_w(z^l, w^l)'$. Acceptance of the newly created solution is assessed based on the following probability:

$$P(accept\ change) = \begin{cases} 1 & \text{if} \quad \Delta s \leq 0 \\ e^{-\Delta s}/B\dot{T} & \text{if} \quad \Delta s > 0 \end{cases} \tag{3.45}$$

   The newly created set of solutions $\mathbf{X}'' = (\mathbf{x_1''}, \mathbf{x_2''}, ..., \mathbf{x_N''})$ with its function values set $Z'' = (z_1'', z_2'', ..., z_N'')$ is sorted in an increasing order list $Z'' = [z_1'', z_2'', ..., z_N'']$ and if the minimum temperature is reached $\dot{T} \leq T_{min}$ the final solution for this weights combination w_i^l is found at the first place $z_i = z_1$ in the set. If not, the temperature is decreased according to the cooling schedule (geometric temperature decrease $\ddot{T} = \dot{T} * \Delta T$ is adopted) and this step is repeated.

4. *Allocate* - The final optimal solution for each combination of weights identified in the above step are finally added to the set of optimal solutions creating a final Pareto-optimal set $\mathbf{X} = (\mathbf{x_1}, \mathbf{x_2}, ..., \mathbf{x_{\Omega^D}})$. For each of these solutions (x_i) its functional values across different criteria z_i^l, $\forall l \in L$, $\forall i \in \Omega^D$ represent the basis for the comparison and trade off among different alternatives.

Of course, the number of solutions Ω^D in the Pareto-optimal set corresponds to the number of used weights combinations and should be sufficient to give a "good" approximation of the whole efficient frontier (Ulungu et al., 1999). Nevertheless it increases with the dimension of the problem and the number of criteria and if too large may render a procedure computationally very demanding. Beside the obvious suggestion that the more detailed assessment of the preferences of the decision makers prior to the analysis may help to significantly reduce the size of the possible weights set, a more sophisticated sampling procedure for the creation of weight sets has been suggested and implemented. The Latin Hypercube Sampling technique is used to advance the creation of the Ω plausible collections of weights sets such that there is only one sample point in each weight set across each range r out of M predefined rages with equal probabilities. Such a sampling technique, mainly used for multidimensional distributions, reduces the creation of mutually dominated weight sets and will be explained in the details later on.

3.3.5. Final Solution with the Branch and Bound Method

After development of the procedure for the identification of the Pareto-optimal solutions for one system configuration (i.e. identification of the element's optimal sizes and capacities by identifying optimal flows), it is of the prime importance to expand the procedure to examine different system configurations (i.e. number and position of elements) and to identify the optimal among them, referred as the **final solution**. For a system with n elements that may take two possible discrete states (for example "yes" or "no") the number of possible configurations is 2^n and corresponding *time complexity function*[18] approaches $O(2^n)$. It is obvious that the examination of all instances would be too time consuming and that it is necessary to introduce some algorithm that is capable of reducing of the number of evaluations without omitting the optimal ones. The Branch and Bound method is adopted. It achieves such reduction by dividing the feasible region of a problem into smaller sub-problems. This is well applicable to the network-type of problems, since they can be easily divided into smaller problems on sub-networks.

The *Branch and Bound* method, first suggested by Land (1960), is a tree search strategy which solves combinatorial problems by implicit enumeration of feasible solutions. Depending on their structural dependencies, all feasible solutions are sorted in a tree and the algorithm saves on computation by discarding the nodes of the tree that have no chance of containing a better solution than already identified one (Bertsekas, 1998). In particular, the algorithm checks whether the solution at the current node in the tree (lower bound) exceeds the best available solution found so far (upper bound). If the lower bound does not exceed the upper bound this node is said to be *fathomed*, which means that it and all its descendants nodes (solutions which are further refinement of this solution) are dropped from further consideration. Obviously the structure of the Branch and Bound tree must be such that the descendent nodes can yield only worse solutions than their predecessors. Rather than creating the tree a priori to the algorithm, it's creation along the progress of the algorithm enables to more easily put the configurations that can not yield solutions better than the current one at the descendent positions. In order to explore the whole set of possible configurations, the algorithm used two basic steps: *forward* and *backward* (Kubale and Jackowski, 1985). The forward steps identifies not yet explored nodes (new configurations) while backward steps moves sequentially back to the first not fathomed node if the current node is fathomed. The Branch and Bound algorithm consists of the following main steps:

1. *Initialize* - Create the first system configuration that has the maximum number of elements ($y_{ij} = 1, \forall a_{ij} \in A$) which all have maximum potential sizes and use the Simulated Annealing procedure to determine the optimal solution $\overline{\mathbf{x}}$. Set its function value $\overline{z}$ as initial upper bound.

2. *Branch* - Create a new system configuration by taking out some potential elements ($\exists a_{ij},\ y_{ij} = 0$). For each new configuration the Simulated Annealing procedure is employed to find the minimum cost solution $\underline{\mathbf{x}}$ and its function value $\underline{z}$ is set as lower bound. The procedure remember all already explored configurations and can visit any node of the Branch and Bound tree only ones.

3. *Bound* - If $\underline{z} < \overline{z}$ then $\underline{z}$ becomes new upper bound $\overline{z}$, $\overline{\mathbf{x}}$=$\underline{\mathbf{x}}$ and the procedure branches *forward* form this node. Otherwise, this node is *fathomed*, procedure go *backward* to the first *not-fathomed* node and all configurations that are further refinement of the fathomed node are omitted. The procedure stops when all nodes of the Branch and

[18] for an algorithm (usually iterative) it is a maximal number of elementary operations required to solve any instance of a given problem(Spelberg et al., 2000).

Bound tree (all feasible configurations) are either visited or fathomed. The final best found function value $\overline{z}$ is the last upper bound solution $\overline{\mathbf{x}}$.

For the case of the multi-criteria optimisation, the above described procedure can be combined with the multi-objective extension of the Simulated Annealing algorithm. Of course, it is again necessary to create a set of Pareto-optimal weights combinations for which the optimal solutions is to be identified, before the optimisation run. The procedure is then re-runed for all weights combinations creating a final set of Pareto-optimal system configurations with the identified optimal network flows.

3.4. Method for the Integration of Uncertainty, Risk and Reliability Considerations

The above presented general optimization procedure that can handle planning, design and operative analysis of water supply networks assumes that all input data (e.g. water demands, available supplies, hydraulic parameters, etc.) can be precisely defined. However, many of the input data and parameters are subject not just to their inherent variability, such as the increase in roughness coefficients due to the sedimentation and the deposition in pipes, but also to the high degree of uncertainty, such as the one connected with predicted water demands for some planning period. Similar variability and uncertainty affect not just the input data but also all other planning, design and operation parameters and criteria (e.g. spatial distribution of new demand points, coincidence of pipe outbreaks and fire fighting situations, pumping energy prices growth, etc.), and have to be addressed during the analysis. The recent advances in the risk-oriented approaches for the management of man-made systems, offer new possibilities to develop systems that better suit to the needs and preferences of the users and provide for the additional savings in cost or the minimisation of some other negative effects. In addition, these approaches promote greater transparency of the systems analysis and decision making and could be one of the milestones for the sustainable development of infrastructural systems.

The ability of a system to perform under a variable range of conditions that may occur during its life time, has been for a long time recognised as more important than just the minimisation of the systems costs (Lansey, 1996; Mays, 1996b; Tung, 1996b). The traditional approach to devise reliable systems is to define the standards that a system has to fulfil and then to gradually improve its characteristics until all standards are accomplished for all predefined stress conditions. The aim is to produce a system whose performance are above certain standards at the lowest costs (Grayman, 2005). The standards are codes of practice that define the minimum system performance level and can be defined in terms of minimum delivered flow rates at demand nodes, maximum withdrawal flow rates at supply nodes, minimum and maximum pressures or some others. This approach is very convenient for the type of analysis where both, "worse" stress conditions (loads) and standards that some system has to fulfil (resistance) can be deterministically determined (resistance > loads). This approach is adopted for the analysis of the system behaviour for the case of failure of some component, so called *Component Failure Analysis*.

The variability and uncertainty of the water supply input parameters is very difficult to be deterministically quantified. Therefore the probabilistic quantifications, in which the uncertain knowledge is expressed through some statistical measures such as the probability density distribution, moments of the distribution, etc., are often used. For such defined input parameters (loads), the evaluation of the performance of a system (resistance) has to be expressed in probabilistic terms too. The performance measures of some system alternative is then expressed as the probability that resistance is greater than load (P[resistance>loads]). The acceptance or rejection of the alternative with such performance depends on the risk perception of a decision maker who may be more or less risk prone. Beside probabilistic quantification of the uncertainty, the Stochastic design approach implies a certain level of randomness in evaluation of the performance of a system. Therefore the Stochastic Simulation approach is adopted for the assessment of a system behaviour for the case of variable and uncertain system parameters, so called *Performance Failure Analysis*.

3.4.1. Component Failure Analysis with the Path Restoration Method

Component failure analysis implies the addition of the spare (extra, additional) components and capacities to a system that can provide for a system operation even without completely or partially failed components (Mays, 1989b). The failure of the individual components are taken as individual stresses that a system should sustain (continue to provide services with given standards). The adopted network type representation of water supply systems provides for the possibility to easily identify affected parts of the system and to effectively identify possible compensation sources. Compensation for some failed element of the network (e.g. water pipe, pump station, check valve, etc.) depends on the existence of the alternative paths (routes) and their capacities. The existence of the alternative paths (backup paths) depends on the system layout and the works of Ostfeld and Shamir (1996) and Ostfeld (2005) have already addressed this question based on the the most flexible pair of operation and backup digraphs that yield a first-level system redundancy[19]. The focus in this study is on the selection and optimisation of the costs of the spare capacities that provide for the satisfaction of some predefined failure scenarios for an already given network layout.

A method for the addition of the minimum cost spare capacities for some predefined failure scenarios developed by Iraschko et al. (1998); Iraschko and Grover (2000) in the field of telecommunication engineering, referred as the Path Restoration method, has been adopted and accommodated for water supply networks. Rather then identifying only replacement paths between affected nodes, this method is based on the identification of source-to-destination replacement paths for all affected source-to-destination pairs, and is very convenient for the application in water supply networks. Such a*global reconfiguration* approach is not just more effective for the distribution of the spare capacities across the network (Iraschko and Grover, 2000) but it identifies the exact alternative supply nodes and their paths to the affected demand points for each component failure. Moreover, all these alternative paths (restoration paths) are calculated in advance (preplanned) and can be quickly activated in cases of emergencies, failures or accidents.

[19] the existence of at least one alternative path that can transport water to each demand node in a case of failure of any arc of a network

In a more formal way, the path restoration routing for a given failure scenario s that affects F source-destination pairs $x_f^{\pi s} \in \Pi_f^s$, $\forall f \in F$ out of which each can be restored in R source-destination restoration paths $x_{f,r}^{\pi}{}^s \in \Pi_{f,r}^s$, $\forall r \in R$, can be defined as:

$$max\ z = \sum_{f \in \Pi_f^s} \sum_{r \in \Pi_{f,r}^s} x_{f,r}^{\pi}{}^s \quad \forall(s) \tag{3.46}$$

subject to:

$$\begin{array}{lr} \sum_{r \in \Pi_r^s} x_{f,r}^{\pi}{}^s = Q_f^s & \forall(x_{f,r}^{\pi}{}^s \in \Pi_{f,r}^s), \forall(s) \\ \delta_{f,r_i}^s x_{f,r}^{\pi}{}^s \leq \kappa_{spare_i} & \forall(a_i \in A), \forall(x_{f,r}^{\pi}{}^s \in \Pi_{f,r}^s), \forall(s) \\ x_{f,r}^{\pi}{}^s \geq 0 & \forall(x_{f,r}^{\pi}{}^s \in \Pi_{f,r}^s), \forall(s) \end{array} \tag{3.47}$$

where $x_{f,r}^{\pi}{}^s$ is the flow assigned to the r-th restoration path x_r^{π} form failed source-destination path x_f^{π} for failure s, Q_f^s is the total affected flow on failed source-destination pair x_f^{π} for failure s, $\delta_{f,r_i}^s x_{f,r}^{\pi}{}^s = 1$ if arc a_i is on r-th restoration path for failed source-destination pair f in the event of failure s and $\delta_{f,r_i}^s x_{f,r}^{\pi}{}^s = 0$ otherwise, and κ_{spare_i} spare capacity on arc a_i.

For water supply networks an additional constraint had to be added to the above defined problem since the identification of the eligible restoration paths depends on the pressure conditions in a network too. Only the paths on which total head losses for the case of the addition of the restoration flow are less or equal to current total head difference between source and destination node are considered as eligible.

$$\sum_{r \in \Pi_{f,r}^s} \Delta H(\delta_{f,r_i}^s x_{f,r}^{\pi}{}^s) \leq H_s(x_{f,r}^{\pi}{}^s) - H_d(x_{f,r}^{\pi}{}^s) \quad \forall(a_i \in x_{f,r}^{\pi}{}^s), \forall(x_{f,r}^{\pi}{}^s \in \Pi_{f,r}^s), \forall(s) \tag{3.48}$$

where $\Delta H(\delta_{f,r_i}^s x_{f,r}^{\pi}{}^s)$ is the sum of all head losses on the restoration path $x_{f,r}^{\pi}$ for failed source-destination path x_f^{π} in case of failure s and $H_s(x_{f,r}^{\pi}{}^s)$, and $H_d(x_{f,r}^{\pi}{}^s)$ are the total heads at the source and destination node of the same restoration path, respectively.

The algorithm of Iraschko and Grover (2000) was adjusted to handle this addition and the basic steps of the algorithm are:

1. *Reserve Network* -. For each failure scenario s, out of the survived portions of affected (failed) paths $f \in F$ and the rest of a network a, so called, *reserve network* is created. Capacities of the reserve network are equal to the current spare (not used) capacities and this network is together with the current head distribution used to identify all eligible restoration paths $r \in R$ for each failed path f.

2. *Existing Spare Capacities* - Out of all eligible restoration paths R the one with the minimum transport costs $r = i$ for which $Min(DC_{var}(x_{f,i}^{\pi}), \forall i \in R)$ is selected and the amount of flow either equal to its spare capacity or to the total affected flow $x_{f,r}^{\pi}{}^s = Min((\kappa_{spare_i}, \forall a_i \in x_{f,r}^{\pi}{}^s), (Q_f^s))$ is added to it. The total affected flow is reduced for this amount $Q_f^s = Q_f^s - x_{f,r}^{\pi}{}^s$, the used spare capacity is removed from the reserve network and the same step is repeated until total affected flow has been restored $Q_f^s = 0$ or all restoration paths $x_{f,r}^{\pi}{}^s \in \Pi_{f,r}^s$ are used.

3. *New Spare Capacities* - If the total affected flow is not restored $Q^s_f > 0$, out of all eligible restoration paths R the one with the minimum investment and transport costs for the addition of not restored flow Q^s_f is selected $r = i$ for which $Min(C_{fix}(x^{\pi}_{f,i} + Q^s_f) + DC_{var}(x^{\pi}_{f,i} + Q^s_f), \forall i \in R)$ and the amount of flow either equal to its maximal capacity or to the not restored flow is added to it ${x^{\pi}_{f,r}}^s = Min((\kappa_{maxi}, \forall a_i \in {x^{\pi}_{f,r}}^s), (Q^s_f))$. The total affected flow is reduced for this amount $Q^s_f = Q^s_f - {x^{\pi}_{f,r}}^s$ and the step is repeated until the whole affected flow is restored all or restoration paths are expanded to their maximal capacities. On the end the reserve network is re-setted for a new failure state.

On the end, since the existence of the spare capacities should not degrade the normal operation of a system, the incremental addition of the spare capacities, where after each component failure analysis the performance of a system for the normal operation conditions is checked, is suggested. This helps to better assess the effects of the addition of the spare capacities and prevents possible obstructions in normal operation.

3.4.2. Performance Failure Analysis with the Latin Hypercube Sampling Method

As previously stated both, the natural variation and the uncertainty of systems parameters such as demands, supplies, hydraulic properties, etc. have to be implemented into the water supply systems' analysis. The adopted approach express the uncertain knowledge with the probabilistic measures and use the stochastic simulations to assess the performance of a system for a large number of artificially created samples that correspond to the predefined probabilistic parameter's definitions. Since the simulation of the water supply systems performance may be quite computationally demanding it is necessary to reduce the number of simulations or the number of stochastic samples to the smallest possible that can still provide for a good statistical evaluation of a system behaviour. Keeping in mind that the aim is to obtain the knowledge about the system behaviour for the whole range of the possible parameter deviations especially taking into account the highest stress conditions, the technique for the creation of the samples was selected accordingly. For the particular case of selection of individual values intended to yield some knowledge about a population in N -dimensional space, exceedingly sparsely at M points, the Latin Hypercube Sampling (McKay et al., 1979) is selected. Among Quasi-Monte Carlo, Descriptive Sampling and Latin Hypercube Sampling for the Risk and Uncertainty Analysis of system behaviour, Saliby and Pacheco (2002) proved that the latest has the best aggregate performance.

For example if the assessment of the demand variation and uncertainty in water supply networks is to be done, the N -dimensional space is the number of demand points at which the variation may occur and M is the limitation to the number of values that are to be taken at each point. The idea of the Latin Hypercube Sampling is to partition uncertainty range of each variable (dimension) into M intervals on the basis of equal probability by accommodating the borders among intervals in such a way to provide the equal total probability within each interval (McKay et al., 1979). This provide for the coverage of the whole variability or uncertainty range for each variable. Since the points within different intervals are selected based on their own probability distribution function, the initial statistics of a parameter is maintained. In order to provide for the representation of the correlations among different variables (e.g. changes in water demands in different towns often show the same general trends), the Improved Latin Hypercube Sampling (ILHS) of Iman and Shortencarier (1984) is suggested for the selection of M samples of N variables. Its general steps are:

- *Selection* - For each variable D_i, $i = \{1, ..., N\}$ one value from each interval $j = \{1, ..., M\}$ is selected at random with respect to the probability density in the interval $P(D_i^j)$. This means that the selection reflect the height of the density function across the interval and the values under bigger probability density will have higher probability to be selected.

- *Pairing* - The M values obtained for the first variable D_1^j where $j \in \{1, ..., M\}$, are paired in a random manner (permutation of equally likely combinations) with the M values of the second variable D_2^j where $j \in \{1, ..., M\}$, crating M pairs (D_1^k, D_2^l) where $k \in \{1, ..., M\}$, $l \in \{1, ..., M\}$. These pairs are combined in a random manner with D_3 values to obtain M triplets (D_1^k, D_2^l, D_3^q) where $k \in \{1, ..., M\}$, $l \in \{1, ..., M\}$, $q \in \{1, ..., M\}$, and so on, until M N-tuplets are formed.

The ILHS algorithm, allows not just the creation of the sample that follows the predefined single-probability distributions of the uncertain variables but also the creation of the sample that reflects the predefined mutual dependencies among variables (multi-distribution) defined in the form of rank correlation matrix. Basically the Iman and Conover (1982) adaptation of the non-parametric[20] rank correlation[21] technique has been used to adjust the pairing process in order to encompass for the correlation among variables. Since it affect only the second part of the sampling procedure (*pairing*) it provide for the integrity of the original sampling scheme and for the usage of any type of the input distribution function of individual variables. It is based on the premise that the rank correlation is a meaningful way to define dependences among input variables (Iman and Conover, 1982). The authors recognise that although the procedure helps to better represents the joint distribution of the input variables it does not guarantee the matching of the entire joint distribution function of the multivariate input variables. If more complete information about the multivariate input distribution is available it has to be used instead of the rank-correlation (Iman and Conover, 1982). Nevertheless, such information are rarely available.

[20]statistical analysis in which specific distribution assumptions are replaced by very general assumptions (distribution free analysis) (Gibbons and Chakraborti, 2003).

[21]analysis of relationships between different rankings (ordering) on the same set of items (Gibbons and Chakraborti, 2003).

3.4.3. System Performance Calculation and Risk-Oriented Selection of Alternatives

After creation of the samples, the performance of the system for these samples has to be calculated. Although the previously described network algorithms identify the distribution of flows and pressures in water supply networks, they are specifically developed for the optimisation of network characteristics (e.g. layout, capacities, sizes, etc.) and are too cumbersome for the calculation of the water supply network performances. Instead this is much more effectively achieved with, for that purpose specially developed, algorithms, so called *network solvers*. These usually iterative, numerical procedures solve the momentum and continuity equation by adopting either flows or pressures as the prime variable and by correcting the other one until the accuracy limit on both of them is reached. The applied network solver, developed by Gessler et al. (1985) and based on Gessler (1981) network solution method, is basically an adaptation of the method of Cross (1936) which is one of the first appeared techniques for the complete solution of the network flow and pressure distribution problem. Although it is not as efficient as the modern matrix based techniques, it allows for a much easier implementation of the pressure controlling devices and has a very transparent and simple calculation procedure. Due to its simplicity, possibility to deal with large networks and good efficiency it is adopted in this study.

The method of Gessler (1981) takes heads at nodes as the prime variables and set up as many equations as there are nodes with unknown heads. In each iteration, based on the heads from the previous step or initially assumed one, the method calculates the flows and losses in arcs of a network. Since these flows still do not satisfy the continuity equations at nodes, they have to be balanced by solving the linearised continuity equations formulated in the matrix form. The resulting coefficient matrix is always symmetrical and for large networks extremely sparse. The algorithm takes advantage of both of these characteristics and use the calculated flows to gradually adjust the head at nodes such that the flow rates balance. The algorithm proved to have a very good convergence (Gessler, 1981).

The use of the network solver provides for the efficient calculation of the system performance for all created samples of input variables. The calculated performance measures, flows and pressures in the first line, are then statistically evaluated and their statistical measures such as the mean, median, standard deviations, etc. are calculated. This provides the basis for the quantification of the system behaviour under variable or uncertain parameters. The calculated statistical values can be used to define the performance and the reliability of water supply systems. For example the statements like: "for accepted uniformly distributed uncertainty of the water demands within the 25 % deviations from the predicted values, the 10 % of the calculated pressures lays below the minimum value" directly express the consequences of the demands uncertainty to the performance of the system and defines the risk of performance failure. For decision makers such statems can be even more simplified to the: "if the demands vary for 25 % this system alternative will have low pressures at 10 % of nodes".

For such or similarly expressed system quality performances each system alternative can be presented to the decision makers. Then it is up to the decision makers' preferences toward different objectives and criteria and to their risk acceptability toward system quality performance to select one of the offered system configurations. Transparent presentation of the different criteria as well as the simple definition of the systems variability and uncertainty should promote the greater participation of the broader range of decision makers and their better understanding of the offered alternatives. Furthermore the simplicity of the applied algorithms enables for the greater application of the presented methodology in the praxis.

4. Model Development and Application

In order to enable easier use and application of the methodology presented in the previous chapter, a planning, a design and an operation computer model are developed and presented in this chapter. They are accommodated to address the specific issues of water supply planning, design and operation management problem, forming unique tools for the integrative analysis of water supply systems. In order to demonstrate applicability, to test validity and to compare efficiency with already existing models, each of the developed models is applied on two theoretical case studies. The case studies $P1$, $D1$ and $O1$ serve for the demonstration of the purpose and capabilities of the models, while the case studies $P2$, $D2$, $O2$ are more complex and computationally demanding and serve for the comparison of results and performances with results of the already existing models reported in the literature. The discussion of the results as well as the analysis of the models' validity, sensitivity and efficiency is provided for each model.

4.1. Planning Model

The rapid expansion of water supply systems and the recognition of the importance of the integrative consideration of natural environment and human built-in systems, in the last century, substantially added to the complexity of water management studies. Furthermore, greater participation of the involved stakeholders such as consumers and broad public, responsible authorities, water supply practitioners, environmentalists, etc. and more transparent analysis and decision making have become new standards in planning and management of water resources. What follows is an attempt to develop a model that can help to better address these issues in planning of water supply systems.

4.1.1. Characterisation of the Planning Problem

Water supply planning can be generally defined as a set of forethought activities with the aim to provide a supply of water at some region for some future time period (Walski et al., 2003). Beside the provision of sufficient water quantity with an adequate quality to all water users, planning of water supply aims at the environmentally sustainable management of natural water supplies as well as at the compromise based long term management of users' water needs. Integrated consideration of natural and economic aspects of water provision and consideration of the needs and preferences of all stakeholders are the prime prerequisites for the achievement of these goals.

In addition to forecasting available supplies and user demands, O'Neill (1972) identifies the following three fundamental questions that water supply planning studies have to address:

1. Which natural resources should be used and to which extent?
2. To which demand area should the resources be allocated?
3. In what order should the resources be exploited?

Although every planning problem has its specifics and may have different objectives, decision variables, controls and constraints, the general form of the Multi-Objective Minimum Cost Network Flow optimisation problem from the equation 3.43 on page 57. can be used to mathematically formulate the planning problem:

$$min. \quad Z = \{z_w, \forall w = 1...W\}, \quad z_w = \sum_{l \in L} (w^l \sum_{\pi_k \in \Pi} \sum_{a_{ij} \in \pi_k} (DC^l_{var_{ij}}(x^{\pi_k}) + C^l_{fix_{ij}}(x^{\pi_k}) y_{ij})) \tag{4.1}$$

subject to:

$$\begin{array}{lll} \sum\limits_{n_j: a_{ij} \in A} x_{ij} - \sum\limits_{n_j: a_{ji} \in A} x_{ji} = b_j & \forall n_j \in N & \\ \sum\limits_{\pi} \delta^{\pi}_{ij} x^{\pi} \leq \kappa_{ij} y_{ij} & \forall \pi \in \Pi & \\ p_{min_j} \leq p_j \leq p_{max_j} & \forall n_j \in N & \\ y_{ij} = 0 \vee 1 & \forall a_{ij} \in A & \\ \sum\limits_{l=1}^{L} w^l = 1, & w^l \geq 0 & \forall w \in W \end{array} \tag{4.2}$$

that has for an aim the identification of the set Z of the Pareto-optimal[1] system configurations, where each configuration is optimal for one combination of the decision maker's utilities (weights) w^l toward the objectives, such as the minimization of environmental impacts, economic costs or social consequence. The achievement of these objectives is measured through different criteria $l \in L$. Since these criteria have different units, the functional dependencies of each criteria from some decision variable (net-cost functions c) are scaled down to their non-dimensional representatives (unit-functions C) that all have the same range, e.g. $[0, 1]$. The unit-functions are distinguished into the fix $C_{fix}(x)$ and variable $C_{var}(x)$ impacts that stands for the impacts that occur during construction of some water supply system and the ones that occur regularly during the systems exploitation. In order to bring these impacts to the same time point the latter are discounted to their net present value $DC_{var}(x)$. Finally, they are weighted according to decision makers' utilities toward different impacts w in order to obtain total impacts function z_w. The flows on conforming simple paths [2] x^{π_k} are selected as the main decision variable since they can be easily connected with other planning parameters such as withdrawal at sources, transport quantities, delivery at demands, that directly address the stated fundamental questions of the planning studies. As far as the constraints are concerned, beside the satisfaction of continuity equations on arcs and nodes and minimum and maximum pressure values, in order to provide for consistency in comparison among

[1] one for which no other solution exists that will yield an improvement in one objective without causing degradation in at least one other objective (Cohon, 1978)

[2] water flows from an individual source to a demand point

different weight combinations, the sum of weights across all objectives within each weight combination w has to be equal 1. Finally the integer variable y_{ij} is included to enable the application of the same optimisation routine to the analysis of existing and the development of new water supply systems.

4.1.2. Accommodation of the Solution Methodology

The solution technique for the defined planning problem have to be capable of efficiently dealing with the following main tasks:

- creation of all system configurations that could match foreseen demands and supplies,
- identification of the Pareto-set of system configurations for a given set of objectives and criteria.

The generation of alternative water supply configurations represents the core of the planning process and involves searching of a very large number of possible permutations and combinations of sources, transport connections and demand centres with the aim to identify the combinations that satisfy the basic requirements that the supplies can match the demands. Vavasis (1995) proved that such optimization problems are NP-hard[3]. Since, it is unlikely that a polynomially bounded algorithm[4] for an NP-hard problem exists, one can either approximate the problem or use an approximation algorithm. The applied approximation algorithm is obtained by combining the Branch and Bound (Land, 1960) and the Simulated Annealing (Kirkpatrick et al., 1983; Cerny, 1985) method. The first is deliberately developed with the aim to improve the efficiency of the search through problems with exponential time complexity functions ($O(m^n)$) such as the problems of systems with n elements and m possible states (i.e. selection of water supply network configuration and pipe's diameters). The second improves the capability of the algorithm to identify the globally optimal solutions for the complex non-linear problems such as the defined optimisation problem that combine linear, convex, concave and step-wise impact functions.

Furthermore, an unique optimal solution for multi-objective problems does not exists. Only the solutions that are optimal for a given utility (preferences) toward different objectives can be identified. Since the utilities toward objectives may significantly influence the direction of the optimisation procedure, there are integrated in the problem formulation in the form of weights and the search for the optimal solution is repeated for the broad range of weight combinations. The solutions for which improvements on individual criteria can not be achieved without degradation in some other, called Pareto-optimal, are the ones that represent the optimal alternatives that are to be presented to decision makers. In contrary to the approaches that first identify the system configurations and then evaluate them for some combinations

[3] a NP-hard problem H is at least as hard to solve as any other problem L for which exists polynomial reduction $L \infty H, \forall L \in NP$ where NP is the class of problems for which a guessed solution can be verified in polynomial time (Spelberg et al., 2000)

[4] one with the polynomial time complexity function $O(f(n))$, where $f(n)$ denotes the maximum number of elementary operations required to solve any instance of the problem

of decision maker's utilities, the applied approach prevents the selection of a sub-optimal solution in a decision making process and provides for enough space to make good trade-off among conflicting objectives. For complex water resource management problems with numeral opposite interests, such decision support is very valuable.

The solution procedure implemented in the planning model consists of the following main steps (Figure 4.1):

1. *Input* - Beside basic water supply network data, such as existing layout and capacities, maximum available water amount at sources, predicted consumer demands, etc., the data for the potential elements such as position, discrete set of possible sizes and capacities and unit fix and variable impacts functions have to be defined. The parameters for the discounting of the variable impacts to their net present value (i.e. time period and interest rates) have to be defined, too.

2. *Initial solution - any feasible flow vector* - First, all potential elements are added to the existing systems with their maximum capacities. The virtual, so called slack, nodes and arcs that provide for the feasibility of the network flows by accepting surplus and providing insufficient flows, are also added. The Maximum Feasible Flow graph procedure that is based on the iterative allocation of maximum flows on paths between source and demand nodes, is employed to identify the flow vector that satisfy all demands and do not violate capacity constraints (Jensen, 1980). This is first, so called initial, solution that does not incorporate the impacts and performance of the network but serves only to prove the feasibility of a system to satisfy water demands for some planning period.

3. *Primal solutions - single-objective solutions* - The Branch and Bound algorithm is used to consider different combinations of potential elements and the Simulated Annealing to identify the minimum impacts flow for each of these configurations by randomly generating new flow vectors, defining corresponding system elements, calculating their impacts and accepting or rejecting them based on the Metropolis-Hastings algorithm (Metropolis et al., 1953; Hastings, 1970). The optimal solution for each configuration (upper bound) is than compared with the, until that point, best found one (lower bound) and if better than it becomes a new lower bound and the further refinement of this configuration are then explored. If this configuration yield a worse solution than already found, the algorithm returns one step back and search another not explored configurations. Since weights toward all criteria are set up to maximum value of 1 such identified solution, called primal, serves only as a reference point for the multi-objective solutions.

4. *Final solutions - multi-objective solutions* - In order to identify the Pareto-front of optimal system configurations the Multi-Objective Simulated Annealing (Ulungu et al., 1995) method has been applied. It is based on the consecutive use of the Simulated Annealing procedure for different set of weights toward different criteria. In order to advance the creation of the Pareto-optimal weight sets the Improved Latin Hypercube Sampling of Iman and Shortencarier (1984) is used. This sampling technique provides for the creation of a sample that cover the whole weight combinations range discretely

sparse with a predefined number of points keeping their predefined probabilistic distribution and mutual rank correlation. The created set of Pareto-optimal configurations that correspond to different possible combinations of decision maker's utilities is called final solution and serves for the trade-off among objectives.

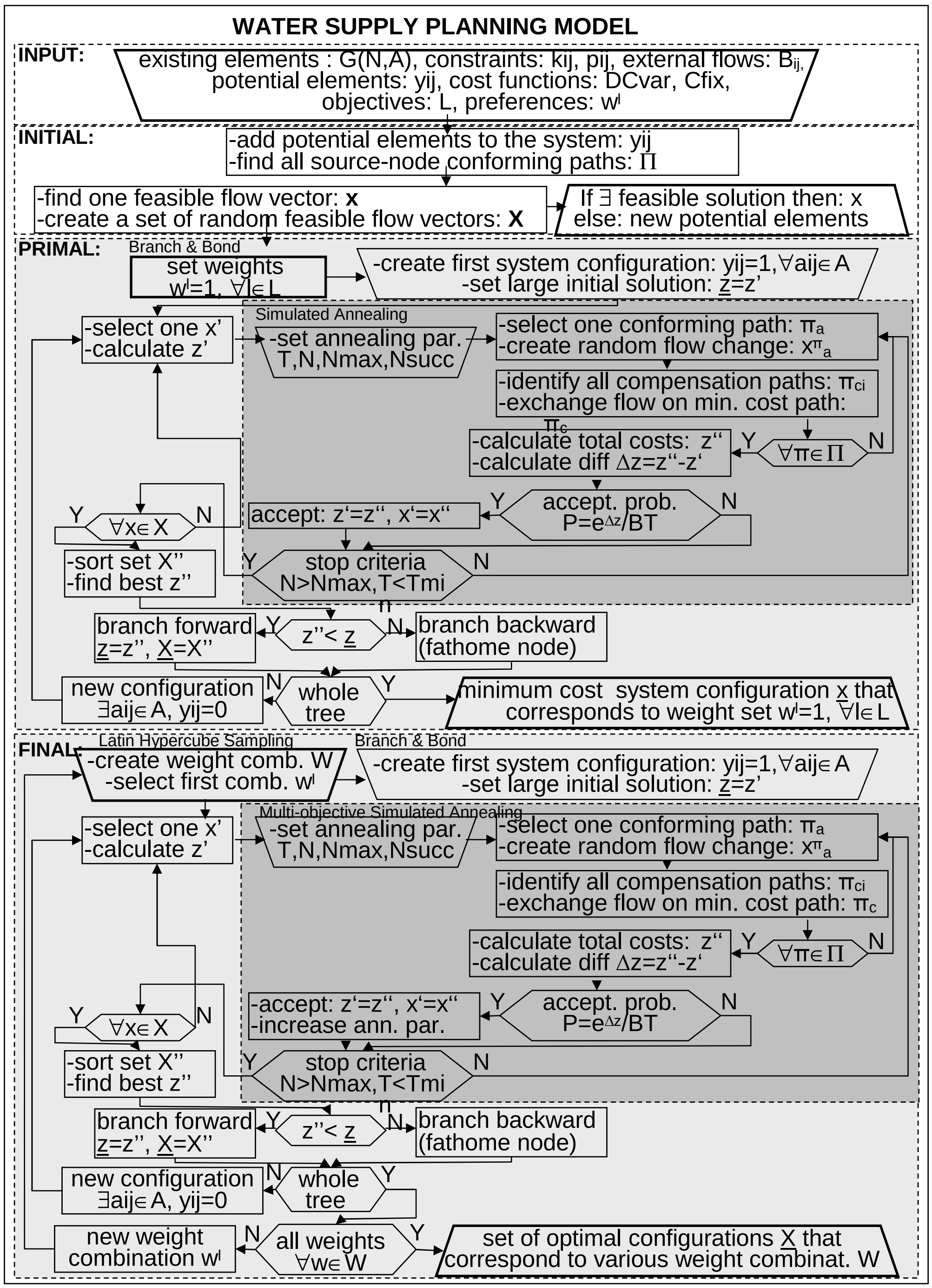


Figure 4.1.: Flow chart of the planning model

4.1.3. Case Study $P1$ - Planning Model Demonstration

In order to illustrate the main purpose of the developed planning model, it is applied at one of the most simple but still one of the most often used case study from the literature. The "2-loop" network of Alperovits and Shamir (1977) presents a standard problem for the optimization of the water distribution networks and had to be slightly modified for the planning study.

Study Description - The original network of Alperovits and Shamir (1977) (circled with dotted line in Figure 4.2) consist of 8 pipes (presented as arrows), 6 demand nodes (presented as trapezoids) and one single river water intake (presented as ellipsoid). In order to re-examine and develop a new water supply strategy for the two demand centres (*N5* and *N7*) for the planning period of 10 years, three potential water sources (*N8*, *N9* and *N10*) with three corresponding transport pipes *A9*, *A10* and *A11* have been considered.

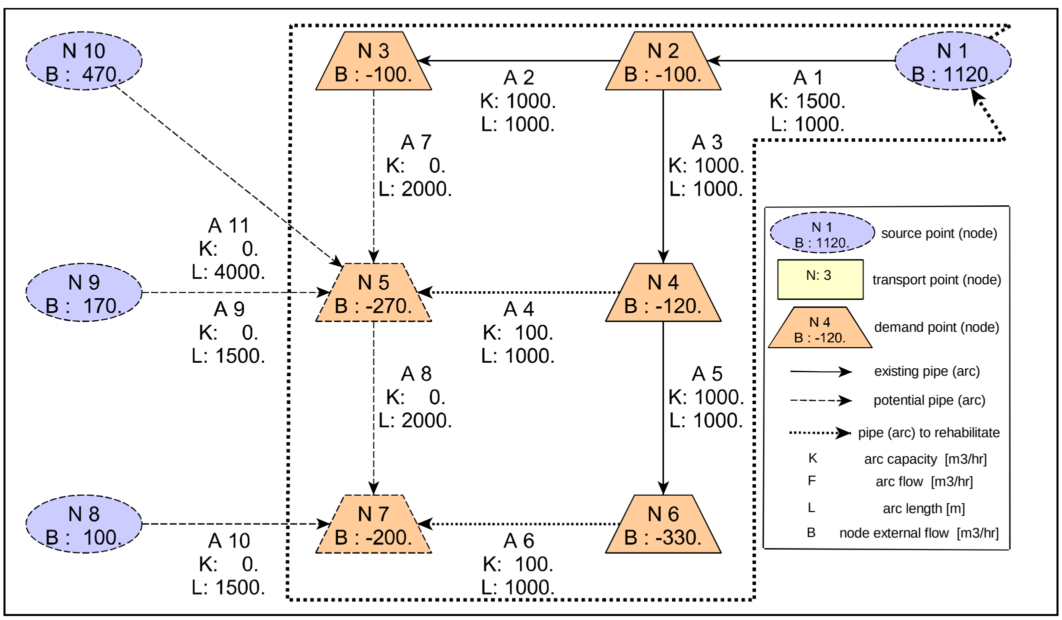


Figure 4.2.: Case study $P1$: Network configuration [adaptation from Alperovits and Shamir (1977)]

It is to be decided whether the supply from the existing river water intake *N1*, from the new spring sources *N8* and *N9* in the vicinity of the demand nodes, from a large regional groundwater aquifer *N10* or some combination among these alternatives is the most optimal planning option considering economic, environmental and social criteria. The already existing river water intake can provide enough water but its treatment is quite expensive and large affected downstream area may cause high environmental impacts. In contrast, the spring and especially the groundwater need less treatment but the investment costs in intake facilities and pipe connections significantly increase total economic costs. Due to the large spatial extent of the groundwater aquifer, it is assumed that the water withdrawals form the groundwater well may cause social disapproval in much more communities than the spring withdrawals and larger social effects are allocated to it. Finally, it is assumed that the existing pipes *A4* and *A6* that supply water to the nodes *N5* and *N7* are old but can be cleaned in order to expand their capacity, while the pipes *A7* and *A8* can be only replaced due to their very bad condition.

Arc ID	Capacity [m^3/hr]		Transport length [m]	Funct. Typ	
	exist.	max.	ecn.cost	fix	var
1	1500	1500	1000	0	7
2	1000	1000	1000	0	7
3	1000	1000	1000	0	7
4	100	1000	1000	1	7
5	1000	1000	1000	0	7
6	100	1000	1000	1	7
7	0	1000	2000	1	7
8	0	1000	2000	1	7
9	0	1000	1500	1	7
10	0	1000	1500	1	7
11	0	1000	4000	1	7

Node ID	Ext.Flow [m^3/hr]	Capacity [m^3/hr]		Aff.area [10^3Ha]	Funct. Typ		Treatment [10^3\$]	Funct. Typ		Aff.comm [nmb.]	Funct. Typ	
org.		exist.	max.	env.cost	fix	var	ecn.cost	fix	var	soc.cost	fix	var
1	1120	1120	1120	90	0	9	1000	4	8	550	1	7
2	-100	100	100	0	0	0	0	0	0	0	0	0
3	-100	100	100	0	0	0	0	0	0	0	0	0
4	-120	120	120	0	0	0	0	0	0	0	0	0
5	-270	0	270	0	1	9	0	0	0	0	0	0
6	-330	330	330	0	0	0	0	0	0	0	0	0
7	-200	0	200	0	1	9	0	0	0	0	0	0
8	100	0	100	5	1	9	200	3	8	23	6	7
9	170	0	170	10	1	9	200	3	8	24	6	7
10	470	0	470	50	1	9	100	3	8	220	0	7

Table 4.1.: Case study *P*1: Characteristics of the network [adaptation from Alperovits and Shamir (1977)]

The characteristics of the network are presented in Table 4.1, where "arc" stands for pipes and "node" for source, demand and transport points (columns *ArcID* and *NodeID*). For each pipe its existing and maximum available water capacities (column *Capacity*) are provided and its economic cost are given with the maximum costs (column *Transport length*) and the form of functional dependency of fixed and variable impacts (column *Func. Typ*) that corresponds to the adopted typical dependencies presented in Figure 3.2 on page 41[5]. Similar to the pipes, the existing and maximum capacity of each source and demand node is given in *Capacity* column. The foreseen water demand and supply availability are presented as external flows to the network (column *Ext.Flow*) where demands are negative and supplies are positive. As far as the economic, environmental and social impacts of the water sources are concerned, they are given through the maximum affected area (column *Aff.area*), the maximum cost for transport and treatment (column *Treatment*) and the maximum number of affected communities (column *Aff.comm*). For each of them the form of functional dependencies for fixed and variable impacts that correspond to the functions from the Figure 3.2 on page 41[6] are given in column *Func. Typ*.

Problem Statement - The problem to be solved, is the identification of the optimal source, or combination of sources, and corresponding transportation arcs that provide for the "optimal" satisfaction of the foreseen demands in the planning period. The optimality is here defined through following three main objectives:

1. Minimize economic costs.
2. Minimize environmental impacts.
3. Minimize communities disapproval.

[5] 0 stands for no dependency, 1 for constant and 7 for linear dependency

[6] 3 and 4 stands for step-wise functions with small and large step increase, 6 and 7 are linear, 8 convex and 9 concave function

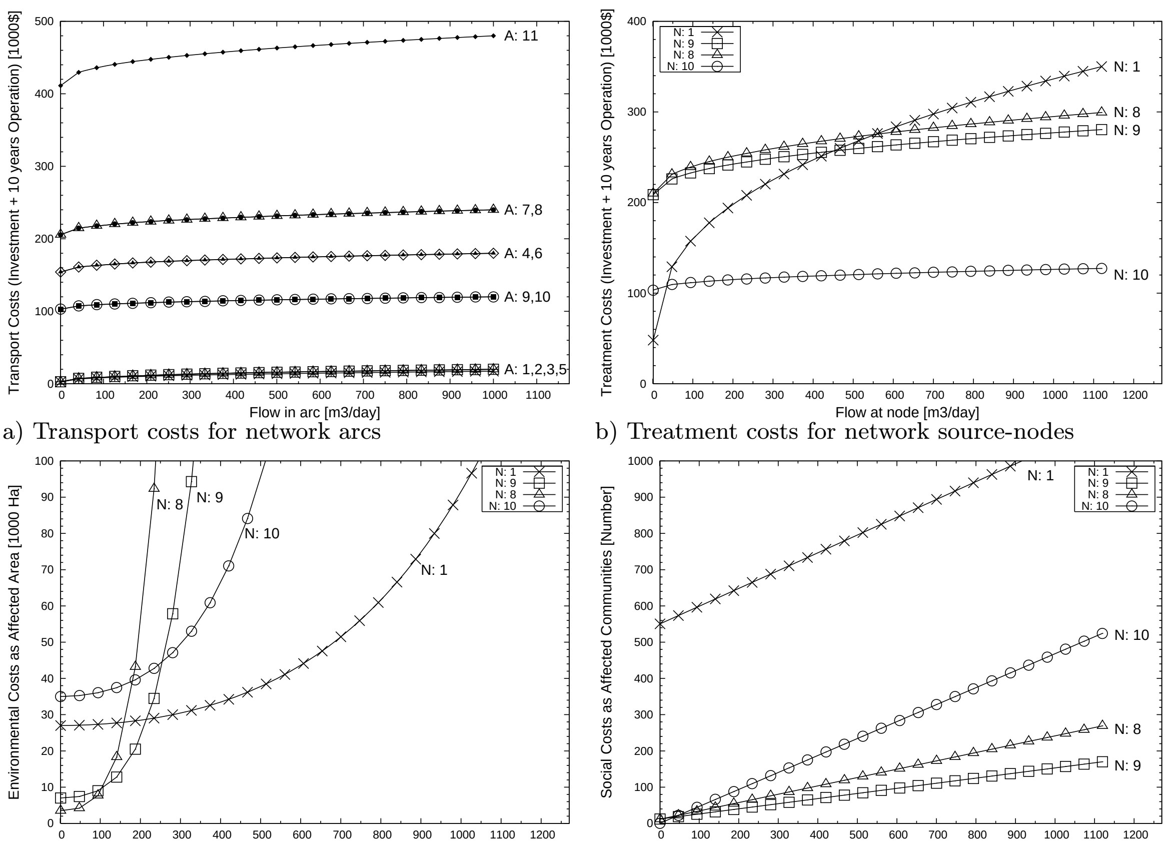
The criteria that measure achievement of the stated objectives and corresponds to the functional relationships that are implemented into the mathematical problem formulation are:

1. Transport costs at each arc and treatment costs at water sources for achievement of the economic objective[7].

2. Affected area from water withdrawal at a source for achievement of the environmental objective.

3. Number of communities that may disapprove with a withdrawal from a source for achievement of the social objective.

The functional dependencies of the criteria from water flow (impact functions), given in Table 4.1, are graphically presented in Figure 4.3. These functions are fictitious but are devised with the aim to present a wide range of different functional forms that may be addressed with the developed model.



a) Transport costs for network arcs

b) Treatment costs for network source-nodes

c) Environmental costs for network source-nodes

d) Social costs for network source-nodes

Figure 4.3.: Case study $P1$: Input economic, environmental and social cost (impact) functions [fictitious]

[7] the third main component of the production costs in water supply, the delivery costs are not considered due to the fact that they are approximately same for each of considered planning options

As it can be seen in graph *a* in Figure 4.3 the operation cost for all pipes are devised as concave functions in order to imitate the "economy of scale" effect. The main economic difference among individual transport pipes comes from the investment costs, which differs from relatively small, for the transport from nearby spring sources (*A10*,*A9*), a bit larger for the rehabilitation of the existing arcs (*A4*,*A6*), proportionally larger for the new arcs (*A7*,*A8*) and very large for the building of new connection (*A11*) to the remote regional groundwater aquifer. For the treatment costs (graph *b* in Figure 4.3) the effect of "economy of scale" is even more present since the investment costs have the same order of magnitude as the operation costs for the adopted planning period of 10 years. For example, although the opening of the new sources (*N8,N9* and *N10*) has larger investment costs than the expansion of the already existing river water intake (*N1*), for river withdrawal amounts larger then 50 m^3/day and 500 m^3/day, the costs of the river water treatment become larger than the investment and operation costs in the new groundwater well and in the new spring sources, respectively.

The environmental impacts of the water withdrawal from different sources (graph *c* in Figure 4.3) are devised in a way that the initially affected areas from the groundwater (N10) and river water (*N1*) intakes are significantly larger than the ones from the springs (*N9*, *N8*). Still due to the very small capacity of the springs their negative environmental impacts progress much more rapidly than for the first two sources. Such exponential dependencies are quite typical for environmental impacts.

Finally the social effects of water withdrawal from some node are presented as linear functions. Due to the already quite large river water use and quite large affected downstream area, many communities are a priory against further expansion of this source (large initial value for line *N1* graph *d* in Figure 4.3). Although initially there is no opposition for the use of groundwater (*N10*), it is assumed that the large use of this strategically important water resources may on a long term cause much larger social discrepancies than the other ones and the largest slope has been allocated to this function. Since there are only a few communities that are affected by the use of water from spring intakes *N9*, *N8* their social impacts are quite small. Finally it has to be underlined again that, although these dependencies aim to imitate the often found conditions in real water supply systems, they are purely fictitious and produced only for the purpose of the illustration of the capabilities of the planning model.

Initial Solution - In order to prove the feasibility of the potential new components to satisfy some given future demands, a feasible flow vector for the system with all existing and potential elements expanded to their maximum capacities is identified. This solution is called initial and it is a first identified flow vector that satisfies network momentum and continuity equations and all network constraints. It does not consider network impacts and serves only as a starting point for the latter iterations of the optimisation algorithm. If one such flow distribution over the water supply system does not exist, the maximum capacity of the potential elements or some new elements have to be increased.

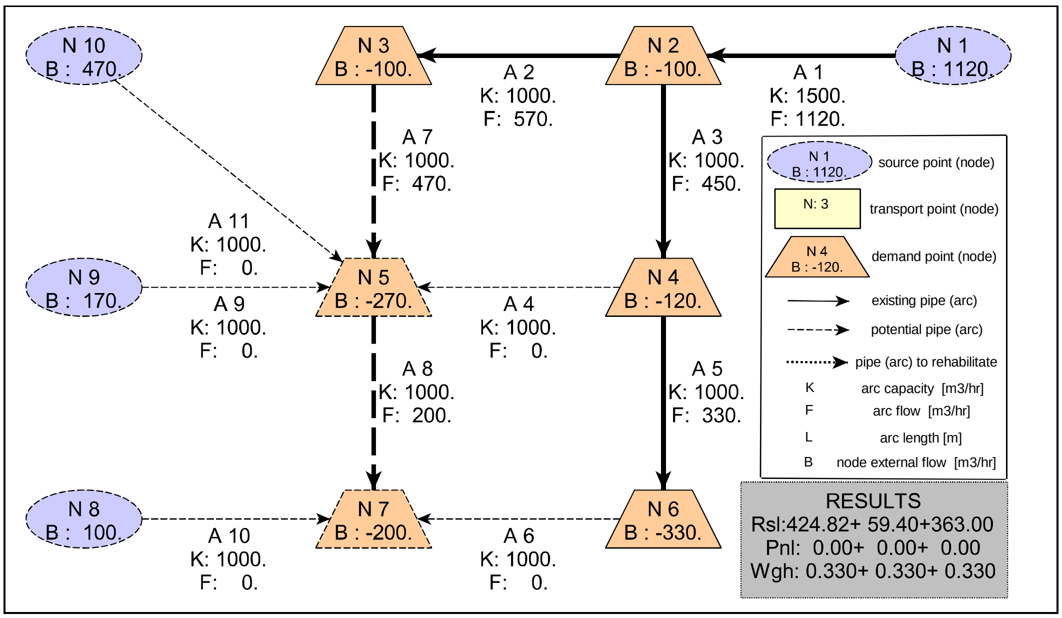


Figure 4.4.: Case study $P1$: Identified initial solution

The identified initial solution for the modified "2-loop" network is shown in Figure 4.4. In this solution, water is supplied to the two demand nodes under consideration (*N5* and *N7*) through potential arcs *A7* and *A8*. If taken into account that the investment costs in replacement of the arcs *A7* and *A8* are much higher than for the rehabilitation of the arcs *A4* and *A6* than it is obvious that this solution is not optimal. Still, it proves the feasibility of the network to supply given demands and serves as a beginning point for the calculation of other solutions. Furthermore, since the Maximum Path Flow algorithm applied allocates flows by adding it on source-demand nodes combinations until the maximum capacity of the corresponding path is reached, it is no wonder that the identified solution uses basically only two paths (form *N1* to *N6* and from *N1* to *N7*) to satisfy all network demands.

Primal Solution - Is a minimum impacts network flow solution obtained using the combination of the Branch and Bound and the Simulated Annealing algorithm to reduce the size or take out some of the, in the previous step added, elements. The utilities toward different objectives and possible trade-off among them are not considered in this solution. The utilities (weight w^l in equation 4.1) toward different criteria all are set up to 1 and different impacts are simply summed to obtain the total function values. The results are then, of course, largely influenced by the impacts with the largest scale. The aim is to produce one solution whose criteria values can be further used for the scaling of each criteria value to the same range. Namely, since the multi-objective solutions will be weighted (scaled according to the decision makers utilities) it is important to have a reference point in order to distinguish weather an improvement in a solution is due to the weights scaling or the real improvement in the flow vector.

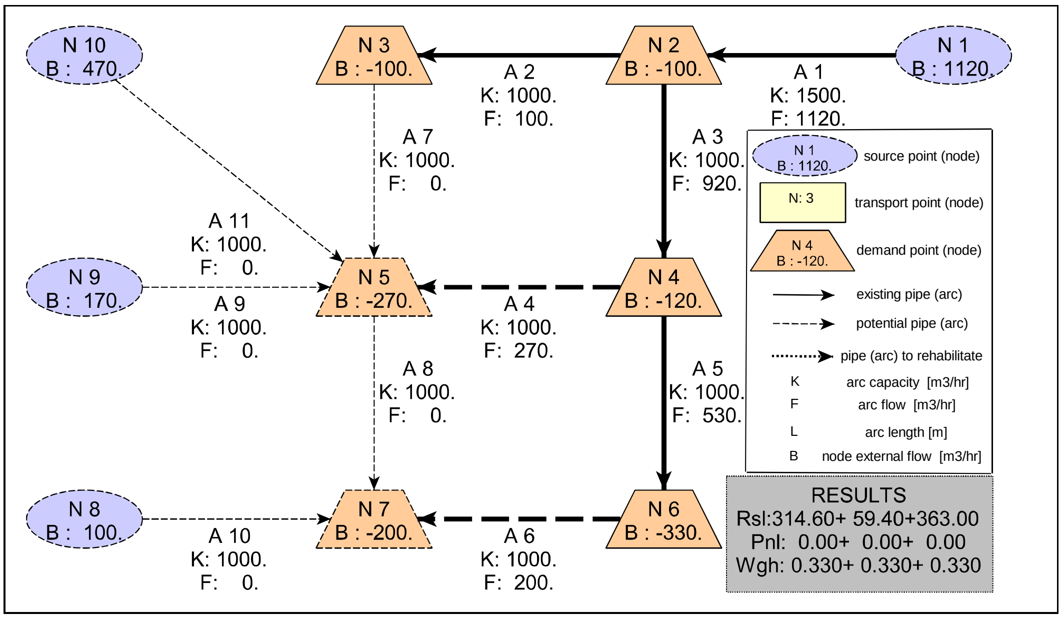
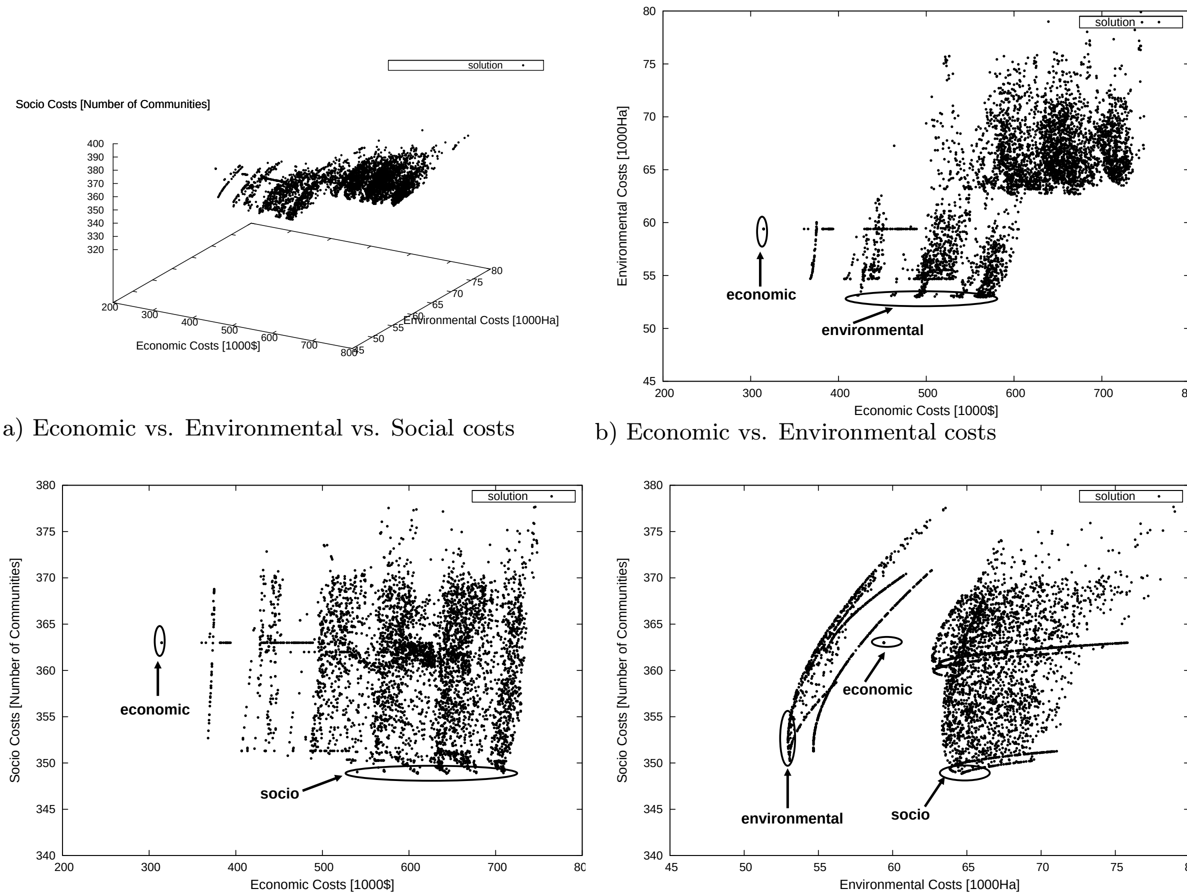


Figure 4.5.: Case study $P1$: Identified primal solution

The obtained primal solution (presented in Figure 4.5) is mainly influenced by the economic impacts since they have the largest scale. Furthermore, since the fixed (investment) costs play a prevailing role, the rehabilitation of the existing pipes *A4* and *A6* is selected as the minimum impact option. This option forwards the use of the existing river water intake and is for sure not the best environmental and social option. Nevertheless the reference environmental and social impact values are obtained.

The presentation of the solution values on individual criteria, obtained during the calculation of the primal solution, in three and two dimensional graphs, provides for the identification of some general dependencies among individual criteria and the identification of possible conflicts among individual objectives. If identified economic, social and environmental criteria values are presented in one graph (graph a in Figure 4.6) a cloud of points that form different frontiers in different objective plains can be observed. Obviously, one single solution that achieves the best across all criteria can not be identified, but instead the optimal solution will be determined by the utilities (weights) toward different criteria. In addition, the grouping of the solutions into smaller clouds, as a consequence of the discrete character of the problem, can be observed. Basically these present different system configurations for which different flow vectors have been created and tested.



a) Economic vs. Environmental vs. Social costs

b) Economic vs. Environmental costs

c) Economic vs. Social costs

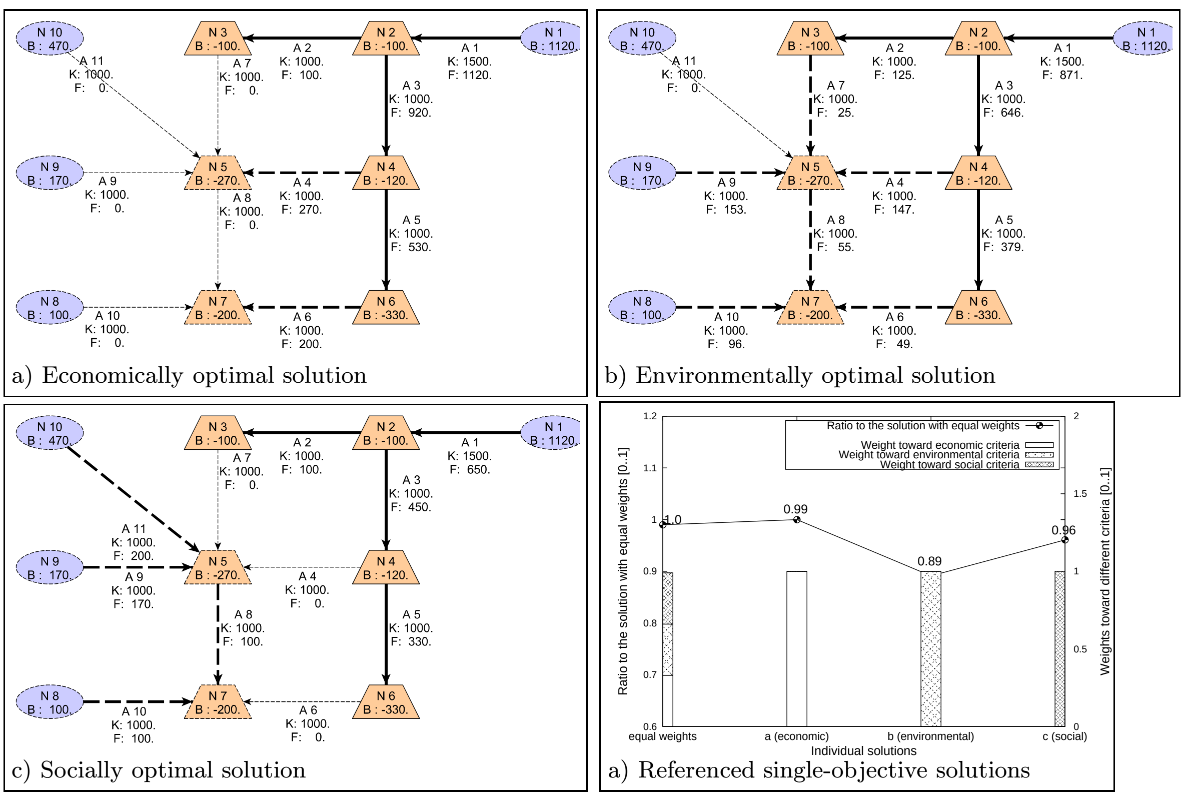
d) Environmental vs. Social costs

Figure 4.6.: Case study $P1$: Obtained values on economic, environmental and social criteria during identification of the primal solution

Presenting the obtained criteria values between environmental and economic criteria (graph *b* in Figure 4.6) and between social and economic criteria (graph *c* in Figure 4.6) shows that there is only one optimum solution from the economic point of view. This solution has been already identified as the primal one. It largely differs from the others since it suggests the rehabilitation of the existing arcs *A4* and *A6* and use of the full capacity of the existing system. Nevertheless for most of the others system configurations, it can be seen that significant improvements on environmental and social impacts can be obtained by better redistributing water withdrawal among sources for slightly higher economic costs. Finally these two graphs show that the minimum environmental and social costs can be obtained with a range of system configurations with different economic costs. The last graph (graph *d* in Figure 4.6) presents identified environmental and social criteria solution values. First to observe is that the most economical solution is far a way from being the most environmentally or socially oriented. For the last two criteria a group of near-optimal solutions can be observed. Since environmental impacts are defined as convex functions from the water withdrawal at sources, these optimal solutions are obtained for a range of configurations that promote distribution of withdrawal across different sources. Similarly the social impacts are defined as linear functions that also promote distributed water withdrawal (see Figure 4.3).

Only, the environmentally oriented solutions tend to use spring sources *N8* and *N9*, while the socially oriented solutions relay on the combined use of spring sources and groundwater source *N10*. As expected, these considerations confirm that each objective has a different solution as the optimal one and that the solution procedure is able to identify a wide range of different solutions.

Single-objective Solutions - Before developing solutions that consider multiple objectives, it is often very usable to first identify the optimal solutions for each objective separately. These solutions form the border of solution space and present the extreme system configurations that would favour only one objective. In addition, such solutions are very valuable for the model validation, since they can be often compared with solutions or expectations obtained by the manual and logical analysis of input data. If the single-objective solutions are consistent with the analytical inspection of the parameter-impact relations, the model will produce sensible results for the multi-objective problem. Of course, such a validation can be done only for simple systems. Still it is often the only circumstantial evidence of the validity of the optimization model results. The single-objective solutions for the defined case study are presented in Figure 4.7.



a) Economically optimal solution
b) Environmentally optimal solution
c) Socially optimal solution
a) Referenced single-objective solutions

Figure 4.7.: Case study $P1$: Identified single-objective solutions (economical, environmental and social) and their improvements relative to the primal solution

As expected the strictly economically oriented solution (picture a in Figure 4.7) suggests the rehabilitation of the existing arc *A4* and *A6* as the optimal option, since the sum of fixed and variable costs for this option is much lower than for the opening of new sources and building of new transport arcs. Although this option promotes further use of the already highly explored river water intake (flow $F = 1120\ m^3/day$ at $A1$) that may have large environmental impacts and high negative social consequence, these two aspects are neglected in this solution. The optimal environmental solution (picture b in Figure 4.7) suggest the use of two new spring sources as the optimal water supply option (flow capacity of $F = 151\ m^3/day$ at $A9$ and $F = 96\ m^3/day$ at $A10$). For low withdrawal values these two sources have very low environmental impacts that favour their selection in this single-objective consideration. Finally the optimal solution from the point of view of the lowest social disapproval (picture c in Figure 4.7) is the one that equally distributes water withdrawal among all sources (flow capacity of $F = 200\ m^3/day$ at $A11$, $F = 170\ m^3/day$ at $A9$ and $F = 100\ m^3/day$ at $A10$). Obviously this option is economically very costly.

In order to test the efficiency of the optimisation procedure to identify the solutions that correspond to the set up objectives, the values across individual criteria of the single-objective solutions are compared to the corresponding values of the primal solution. The obtained results are presented in picture d in Figure 4.7 and present the relative improvement from the primal solution. In addition the economic, environmental and social weights used to obtain these single-objective solutions are presented on the right axis. The obtained ratios show the influence of the used weight combinations on the optimisation procedure. If only the economic objective is favoured ($w^{ecn} = 1, w^{env} = 0, w^{soc} = 0$), the obtained result is the same as for the primal solution. This states again the large influence of the economic costs on the primal solution. In contrast, if environmental and social objectives are favoured ($w^{ecn} = 0, w^{env} = 1, w^{soc} = 0$ and $w^{ecn} = 0, w^{env} = 0, w^{soc} = 1$) than the obtained results improve on these two criteria. Although improvements are not large, they still prove the capability of the optimisation procedure to accommodate its search direction to the given utilities toward different objectives. If known that most of the optimisation methodologies first produce solutions and then try to evaluate their performance using some kind of multi criteria decision making, the previous simple statement questions such an approach. If the optimal solution is to accommodate some predefined utilities toward different objectives, then these utilities have to be encompassed within the optimisation procedure and not after. Unfortunately, since the utilities toward objectives are mainly not available in advance, the developed methodology suggests to identify a large number of optimal solutions that correspond to different combinations of decision maker preferences, in order to provide for a broad range of optimal solutions among which the decision makers can make trade-off.

Mulit-objective Solutions - In order to efficiently identify a broad range of optimal solutions that correspond to various possible combinations of the decision maker's utilities, the Latin Hypercube Sampling method is used to create a set of independent and non-dominated weight combinations. For each weight combination the minimum impacts solution is found. In order to distinguish among the improvements in criteria values in different solutions from the change obtained by scaling with different weights, values across each criteria are referenced to their value in the primal solution - divided with the primal solution value. Obtained ratios are are presented in Figure 4.8. Used weight combinations are presented on the right axis of the same Figure.

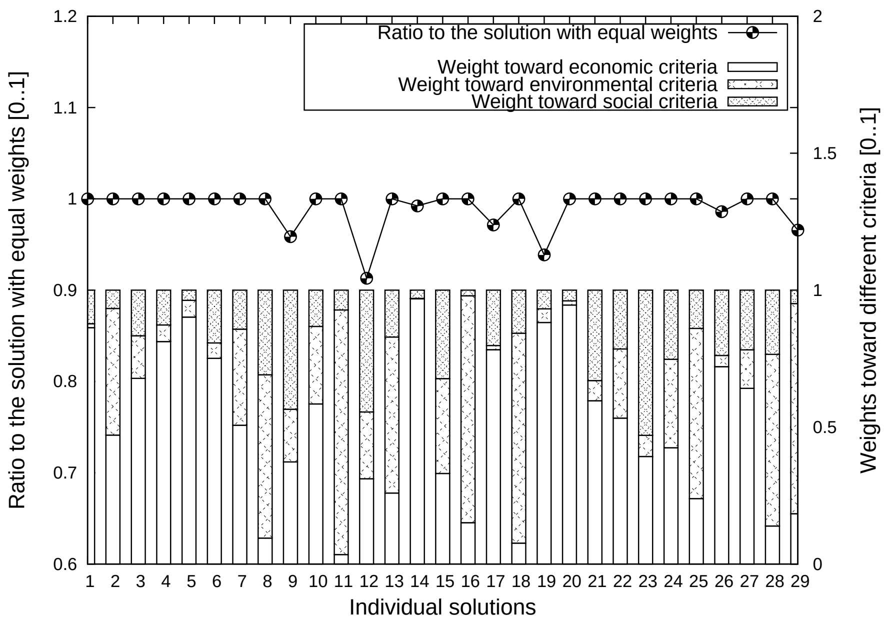
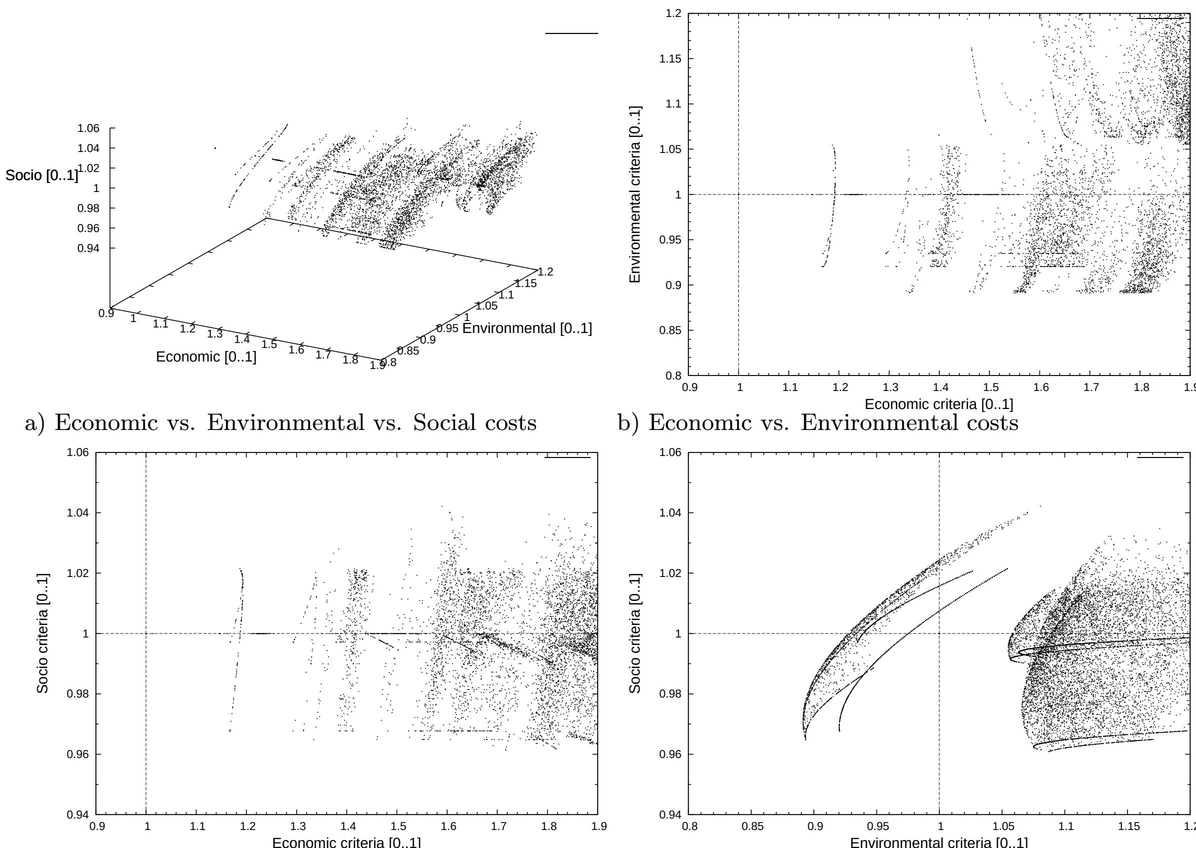


Figure 4.8.: Case study $P1$: Comparison of the multi-objective solutions with the primal one for different weight combinations

It can be seen that the identified solutions are quite similar for most of the weight combinations and only very few ones behave slightly better. Exactly these solutions may be of the special interest for decision makers since they provide additional benefits in some criteria without sacrificing too much on another. Even more, it is not only that the selection of the few optimal ones out of a very large number of possible solutions may significantly improve the decision making process, but it is also that presentation of the utilities toward different criteria (given on the right axes in Figure 4.8) may significantly contribute to the transparency of the whole approach and enable easier trade-off among objectives. It can be seen that the solutions with the large weights put to the environmental and social criteria bring some additional benefits in comparison with the primal solution.

Finally the solution values on economic, environmental and social criteria obtained during the identification of the optimal multi-objective solutions are presented in Figure 4.9. For all three criteria presented together (graph a in Figure 4.9) a cloud of solutions is again formed. It states that there is no individual solution that is the best across all criteria. If this cloud of points is sectioned on individual 2D plains (graphs b, c and d in Figure 4.9) then the relations among individual criteria show similar behaviour as for the primal solution. Only this time the focus is on the identification of the solutions that achieve better than the primal solution on more than one criteria. All solutions that are left or below the horizontal and vertical line through (1,0) and (0,1) point achieve better than the primal. Since the primal solution is optimised for the economic cost, the better solution on this criteria can not be found. In contrast, there has been a whole range of solutions that behave better on environmental and social criteria. The improvements of about 10% on envoronmental and arround 5% on social critera may be achieved by sacrifising the economic criteria for about 10%.



a) Economic vs. Environmental vs. Social costs b) Economic vs. Environmental costs

c) Economic vs. Social costs d) Environmental vs. Social costs

Figure 4.9.: Case study $P1$: Obtained values on economic, environmental and social criteria (relative to the primal solution) during identification of the multi-objective solutions

4.1.4. Case Study $P2$ - Planning Model Validation

Since there are no universal test cases on which the efficiency of different water supply planning models can be compared, the case study developed by Vink and Schot (2002) is selected as representative. This study also has as a prime aim the integration of multiple objectives in water supply system analysis and is based on the approximate solution technique (i.e. Genetic Algorithm). By trading off among economic, environmental and social objectives for interdependent and non-linear drawdown related criteria such as economic costs, agricultural yield reduction, energy consumption, ecological damage to wetland vegetation and social perception of the use of strategic groundwater reserves, it searches for the optimal drinking water production configuration among different ground and surface water sources. In addition it compares the accuracy of the results produced with the approximate procedure (i.e. Genetic Algorithm) with the analytical results and compares its efficiency with the efficiency of the Stochastic Simulation models (i.e. Monte Carlo Simulation).

Study Description - The case study of Vink and Schot (2002) is a fictitious one inspired by a water supply system located in the south of the Netherlands that consists of 10 production wells interconnected with a transport network (Figure 4.10). Some wells (presented as ellipsoids) pump deep groundwater (*N1*,*N3*,*N5*), other pump out of relatively shallow aquifers (*N2*,*N4*,*N7*,*N8*,*N9*,*N10*) and one uses the water directly from a river (*N6*). The water is transported to the urban zones (presented as trapezoids) with the demand defined on an annual basis expressed in 10^6 m^3. The water in shallow aquifers is of relatively poor quality, owing to agricultural production, and requires extensive purification. Although the water from the river intake also need extensive purification, its pumping invokes very little drawdown and therefore no damage to wetland vegetation. At several locations wetland vegetation is highly dependent on the groundwater level (*N1*,*N3*,*N4*,*N9*,*N10*). In addition, the suitability of the deep groundwater for drinking water production is excellent, but extensive use would result in the depletion of groundwater reserves that are considered as the strategic water sources and is from a society point of view not desirable. The position of water sources and water demands with their external flow values as well as the lengths and capacities of the interconnections, are schematically presented in Figure 4.10.

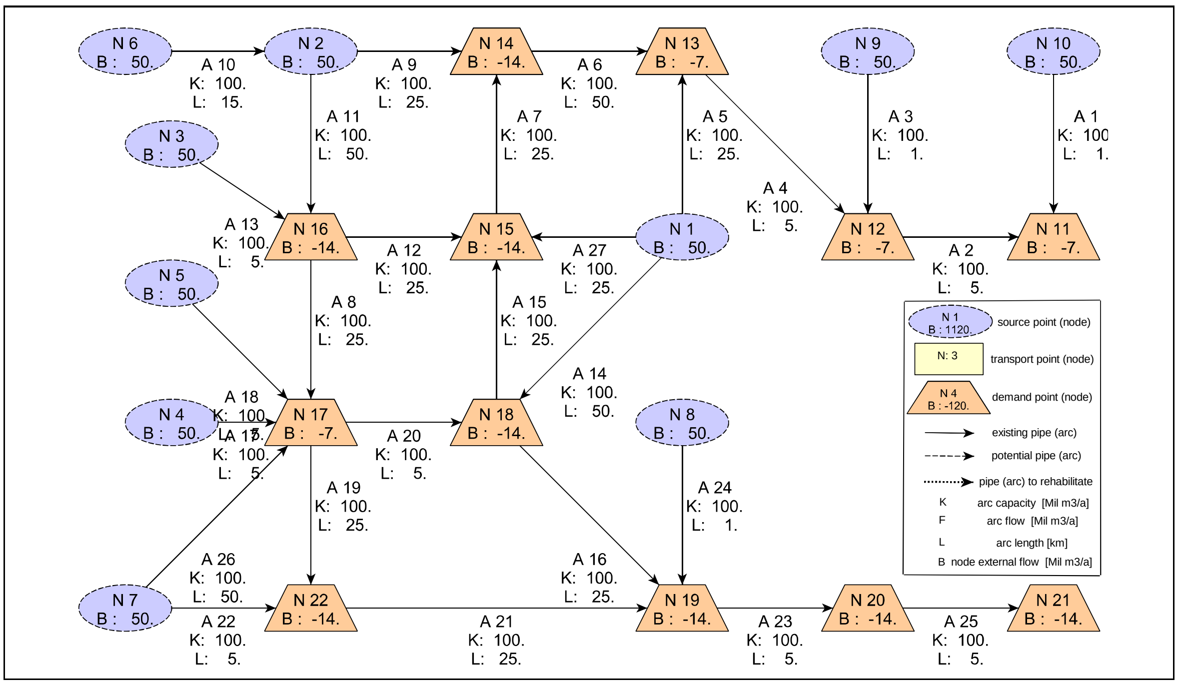


Figure 4.10.: Case study *P*2: Network configuration [adaptation from Vink and Schot (2002)]

The characteristics of the network arcs and nodes with the original identification numbers are presented in Table 4.2 in columns *ArcID* and *NodeID*. Anticipated water demands (column *Ext.Flow*) for the planning period of 10 years as well as the maximum capacities of existing and new water sources (column *Capacity*) are also given. As far as the economic, environmental and social impacts are concerned they are given as maximum costs for transport (column *Transport*) and treatment (column *Purification*), damaged vegetation area (column *Veg.damag.*) and socially negative preferences (column *Soc.Pref.*). For each of them the form of functional dependency of fixed and variable impacts from the flow (column *Func. Typ*) that corresponds to the adopted typical dependencies presented in Figure 3.2 on page 41 is given[8].

Arc ID	Capacity [$10^6 m^3/a$] exist.	Capacity [$10^6 m^3/a$] max.	Transport len. [km] ecn.cost	Funct. Typ fix	Funct. Typ var
1	100	100	1	0	7
2	100	100	5	0	7
3	100	100	1	0	7
4	100	100	5	0	7
5	100	100	25	0	7
6	100	100	50	0	7
7	100	100	25	0	7
8	100	100	25	0	7
9	100	100	25	0	7
10	100	100	15	0	7
11	100	100	50	0	7
12	100	100	25	0	7
13	100	100	5	0	7
14	100	100	50	0	7
15	100	100	25	0	7
16	100	100	25	0	7
17	100	100	5	0	7
18	100	100	5	0	7
19	100	100	25	0	7
20	100	100	5	0	7
21	100	100	25	0	7
22	100	100	5	0	7
23	100	100	5	0	7
24	100	100	1	0	7
25	100	100	5	0	7
26	100	100	50	0	7
27	100	100	25	0	7

Node ID	Ext.Flow [10^6m3/a	Capacity [$10^6 m^3/a$] exist.	Capacity [$10^6 m^3/a$] max.	Veg.damag. [Ha] env.cost	Funct. Typ fix	Funct. Typ var	Purification [10^6\$] ecn.cost	Funct. Typ fix	Funct. Typ var	Soc.Pref. [nmb.] soc.cost	Funct. Typ fix	Funct. Typ var
1	50	50	50	17000	1	8	0.2	0	7	100	0	7
2	50	50	50	300	1	8	1.9	0	7	80	0	7
3	50	50	50	10000	1	8	0.1	0	7	100	0	7
4	50	50	50	15000	1	8	0.5	0	7	70	0	7
5	50	50	50	8000	1	8	0.3	0	7	100	0	7
6	50	50	50	300	1	8	3.6	0	7	80	0	7
7	50	50	50	250	1	8	1.9	0	7	90	0	7
8	50	50	50	450	1	8	1.9	0	7	90	0	7
9	50	50	50	20000	1	8	1.9	0	7	90	0	7
10	50	50	50	15000	1	8	1.9	0	7	90	0	7
11	-7	7	7	0	0	0	0	0	0	0	0	0
12	-7	7	7	0	0	0	0	0	0	0	0	0
13	-7	7	7	0	0	0	0	0	0	0	0	0
14	-14	14	14	0	0	0	0	0	0	0	0	0
15	-14	14	14	0	0	0	0	0	0	0	0	0
16	-14	14	14	0	0	0	0	0	0	0	0	0
17	-7	7	7	0	0	0	0	0	0	0	0	0
18	-14	14	14	0	0	0	0	0	0	0	0	0
19	-14	14	14	0	0	0	0	0	0	0	0	0
20	-14	14	14	0	0	0	0	0	0	0	0	0
21	-14	14	14	0	0	0	0	0	0	0	0	0
22	-14	14	14	0	0	0	0	0	0	0	0	0

Table 4.2.: Case study $P2$: Characteristics of the network (adaptation from Vink and Schot (2002))

Problem Statement - The problem to be solved is the distribution of the discharge rates over the available wells in such a manner that the total of adverse impacts is minimal. Whereby the economic, environmental and social objectives are stated through following criteria:

1. Minimize total economic costs.
2. Minimize damage to wetland vegetation.
3. Minimize negative social discrepancy.

These criteria are defined as the functional dependencies of the impacts from the discharge rate and imitate vegetation degradation, purification costs and social discrepancy of water withdrawal. "The vegetation degradation is assessed by a symbolic non-linear impact model, using a distributed approach of drawdown and fictitious, location-specific, data on vulnerability to drawdown and value of vegetation, and the purification and transport costs are adopted as linear functions of discharge" (Vink and Schot, 2002). Vegetation damage and purification costs as a function of the flow rate are presented in Figure 4.11.

[8] 0 stands for no dependency, 1 for constant, 7 for linear and 8 for convex dependency

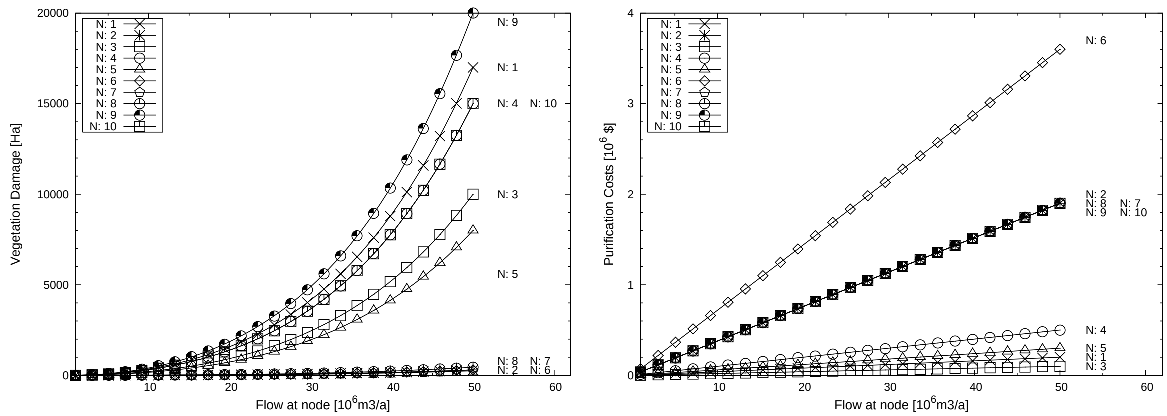


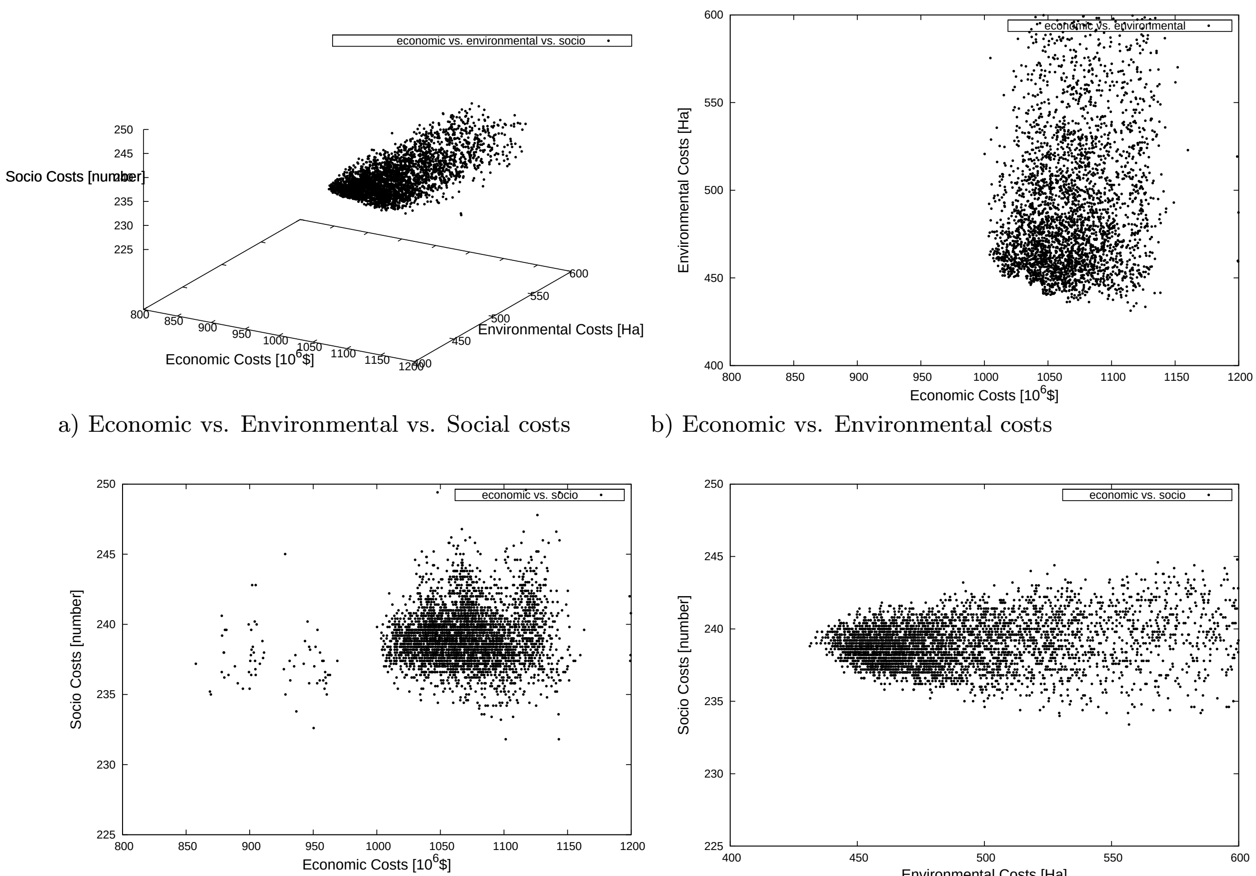
Figure 4.11.: Case Study $P2$: Input vegetation damage and purification cost functions [Vink and Schot (2002)]

Model Validation - In order to validate the model, the optimal solutions for each individual objective are produced and the results are presented in Table 4.3. These solutions enable comparison with the analytical inspection of the discharge impact relations and simple verification of their plausibility. As it can be seen the best economic solution (column *Economic*) allocates the majority of the water withdrawal to the deep groundwater wells (*N1*, *N3*, *N5*) in order to reduce on purification costs. Furthermore, the shallow aquifer well (*N8*) is used to cover water demands in the southern part of the network in order to avoid too large transport costs. In contrary, the best social solution (column *Socio*), minimise the use of groundwater, due to the predefined long term importance of it, and identify the combination of shallow and river water extraction as the best exploitation strategy (*N2*, *N4*, *N6*). Finally, the exclusively environmentally oriented solution (column *Environment*) tend to redistribute withdrawal toward less environmentally damaging wells (*N2*, *N6*, *N7*, *N8*) but still keep the distributed water withdrawal among all water wells as an effect of the concave dependencies among vegetation damage and the withdrawal rate that favour low withdrawal at all sources.

Node ID		Ext.Flow	Capacity		Feasible	Primal	Economic	Environment.	Socio
		$[10^6 m3/a]$	$[10^6 m^3/a]$		$[10^6 m3/a]$	$[10^6 m3/a]$	$[10^6 m3/a]$	$[10^6 m3/a]$	$[10^6 m3/a]$
org.	virt.		exist.	max.	withdrawal	withdrawal	withdrawal	withdrawal	withdrawal
1	2	50	50	50	**50**	0	**33**	1	0
2	4	50	50	50	**50**	**35**	0	**34**	**48**
3	6	50	50	50	**40**	0	**22**	2	0
4	8	50	50	50	0	0	0	1	**50**
5	10	50	50	50	0	0	**46**	2	0
6	12	50	50	50	0	**35**	0	**34**	**42**
7	14	50	50	50	0	**40**	0	**37**	0
8	16	50	50	50	0	**30**	39	**27**	0
9	18	50	50	50	0	0	0	1	0
10	20	50	50	50	0	0	0	1	0
SUM					140	140	140	140	140

Table 4.3.: Case study $P2$: Identified initial, primal and single-objective solutions

All identified results are as expected and align with the analytical investigation of the input data. They completely align with the results of Vink and Schot (2002). In addition, in Table 4.3 the initial and the primal solutions are also presented. It can be seen that the initial solution is just the first one that satisfies the total sum of demands ($\Sigma = 140\ 10^6 m^3$) from the first available wells (*N1*, *N2*, *N3*). The primal solution is a not weighted sum of single-objective solutions and is governed by the criteria of the largest scale. In this case the environmental criteria has the largest scale and influence the primal solution predominantly.



a) Economic vs. Environmental vs. Social costs

b) Economic vs. Environmental costs

c) Economic vs. Social costs

d) Environmental vs. Social costs

Figure 4.12.: Case study $P2$: Obtained values on economic, environmental and social criteria during identification of the primal solution

The presented values on individual criteria, obtained during the calculation of the primal solution, in three and two dimensional graphs, illustrate how difficult is to find the compromise among different criteria. As it can be seen in graph a in Figure 4.12 a cloud of points is formed. Looking closely at the dependencies among individual objectives, the formation of the Pareto-fronts among economic toward environmental and environmental toward social criteria can be observed (graphs b and d in Figure 4.12). Looking at graph c in the same figure, a number of solutions that have much lower economic costs for approximately the same social impacts can be observed. These relay on the large exploitation of the shallow groundwater aquifers that are near to the consumption centres and are from a social point of view perceived as less strategically important than deep aquifers. Nevertheless these solutions cause so large vegetation damages that are out of the scale selected for graph *b* in which the range of optimal environmental solution is.

Model Sensitivity - For the multi-objective optimisation problems the question of "whether a procedure is able to identify a global optimum" transforms to a question of "whether a procedure is able to identify the full range of optimal solutions that correspond to different combinations of utilities toward different objectives". Therefore a set of Pareto-optimal solutions for a set of 29 weight combinations is produced. The values of the obtained solutions on each individual criteria are referenced to the corresponding value from a primal solution in order to bring all solutions to the same scale and to eliminate the influence of the weight on the calculated value. The individual ratios are summed to obtain the total ratio to primal solution which are then together with corresponding weight combinations presented in Figure 4.13.

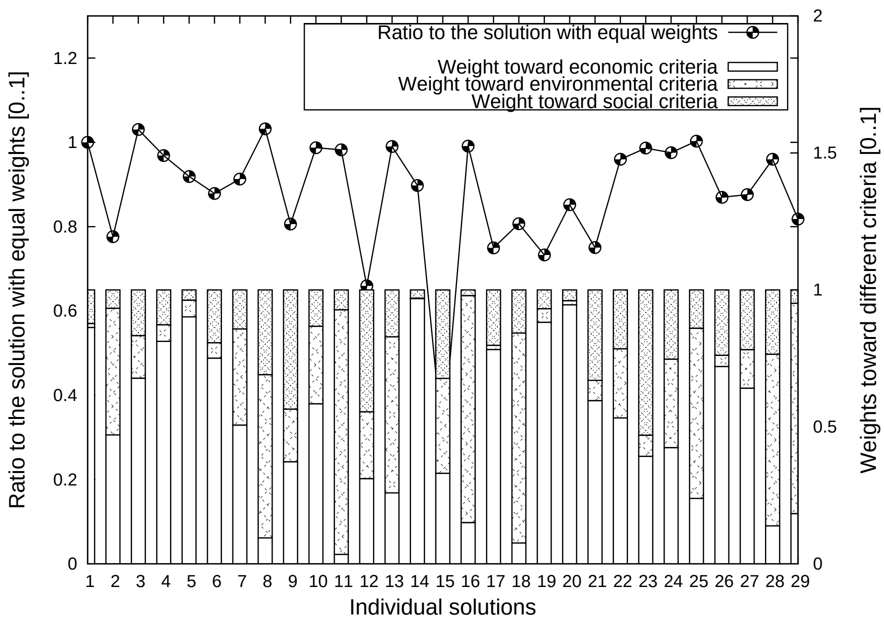
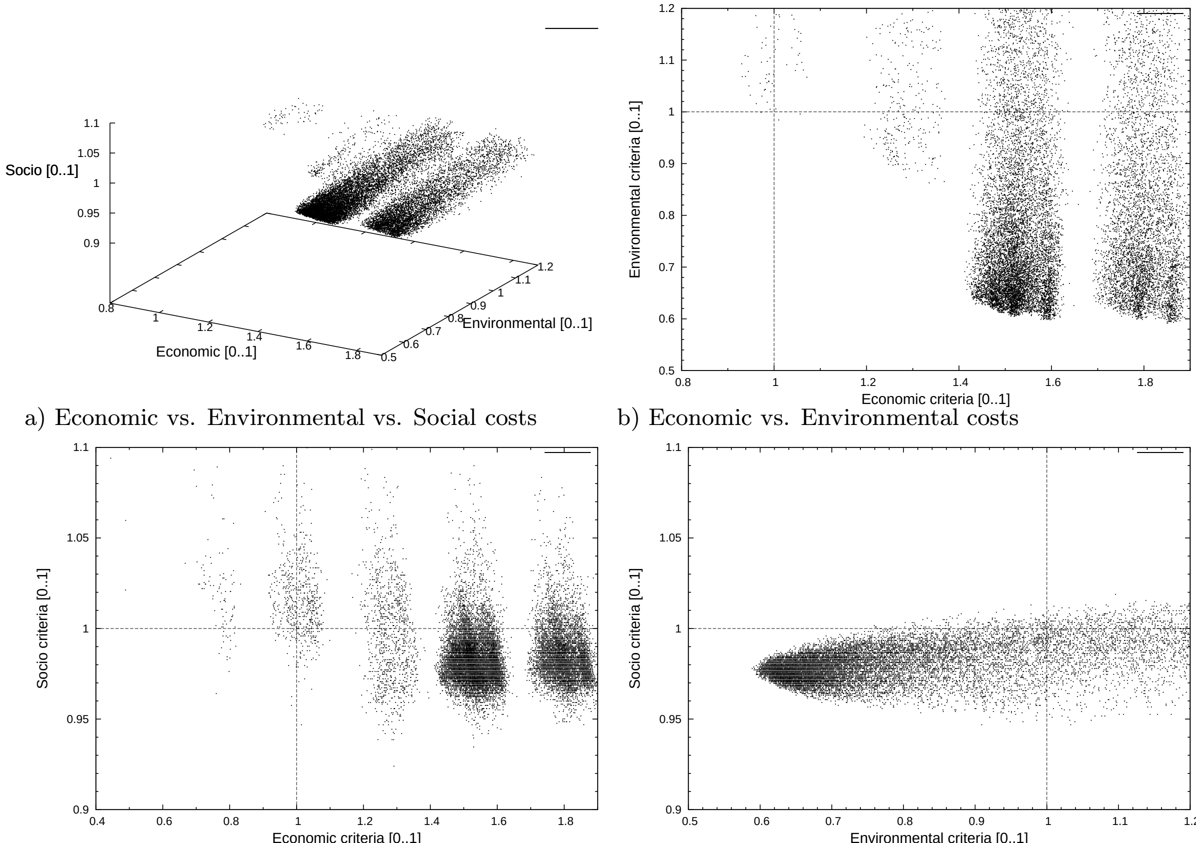


Figure 4.13.: Case study $P2$: Comparison of the multi-objective solutions to the primal one for different weight combinations

All solutions in Figure 4.13 that have ratio smaller then 1 present the improvements from the primal solution. It can be noticed that the improvements in total solution value are achieved for the different weight combinations proving that the optimisation procedure is able to identify optimal solutions that correspond to different weight combinations. At the same time this proves that the weights are the most sensitive parameter of the optimisation procedure and that the identified final solution is highly dependent on the predefined weight combinations. Therefore it is very important to define such a set of weight combination that will enable the identification of the Pareto-optimal set of solutions that are going to be acceptable for the decision makers. If the information about the preferences of the decision makers are not available in advance than the set of weight combinations should cover the whole range of the possible variations among preferences.



a) Economic vs. Environmental vs. Social costs
b) Economic vs. Environmental costs
c) Economic vs. Social costs
d) Environmental vs. Social costs

Figure 4.14.: Case study $P2$: Obtained values on economic, environmental and social criteria (relative to the primal solution) during identification of the multi-objective solutions

In addition, the visualisation of the obtained individual criteria values during the multi-objective optimisation procedure, such as in Figure 4.14, enables for the better analysis of the dependences among different objectives and could be very beneficial in making trade-offs among objectives. On the 3D presentation (graph a in Figure 4.14) the discrete nature of the problem prevent the formation of smooth Pareto-fronts, but instead the solutions are grouped into few clouds with similar criteria values. In order to examine relations among these distributed solutions, the 2D graphs (b, c and d) are given. Again the solutions that achieve better than the primal one on more than one criteria are sought. As it can be seen there is a very small number of solutions that perform better than the primal form the economic point of view (solutions that are within the 0,0 to 1,1 quadrant on graphs b and c in Figure 4.14). In contrast, significant improvements in environmental and social criteria can be achieved (graphs c and d) but only on the costs of economic criteria. It is to notice, that the identified optimal solutions lie in a much broader range then the ones identified with only one weight combination and provide much more space for trade-offs among individual objectives.

Beside given weight combinations the model is obviously sensitive to the parameters of the

Simulated Annealing. Although this is a very robust optimisation procedure, the selection of the temperature decrease, the number of allowed maximal and successful iterations at each temperature level and the constant that relates the temperature to the function value significantly influence the convergence and duration of the procedure. For the previously presented two case studies the progress of the optimisation procedure is presented in Figure 4.15. As typical for the Simulated Annealing, the procedure oscillates, first in larger and than in smaller steps, until it reaches the optimal solution. Since it is an optimisation procedure with inherent randomness, the best solutions may be created even at the beginning of the optimisation and not only at the end.

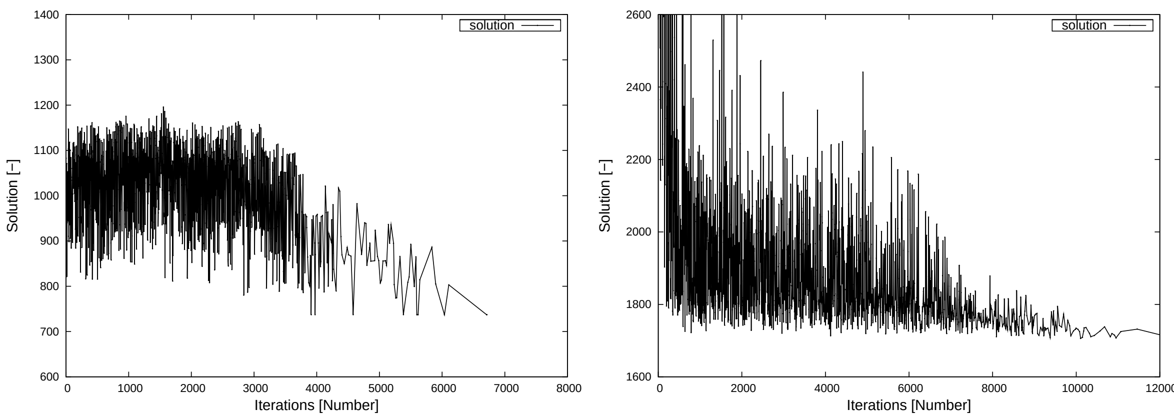


Figure 4.15.: Case Study $P2$: Progress of the optimisation for the case studies *P1* and *P2*

In order to present the effects of the implemented multi-objective extension of the Simulated Annealing (MOSA) the progress of the algorithm according to the individual objectives and accounted impacts on their criteria is presented in (Figure 4.16). It can be noticed that the optimal solution is reached by gradual improvements on all objectives and not only one. This proves the true multi-objective nature of the suggested methodology and its ability to deal with the objectives and criteria with different units and scales.

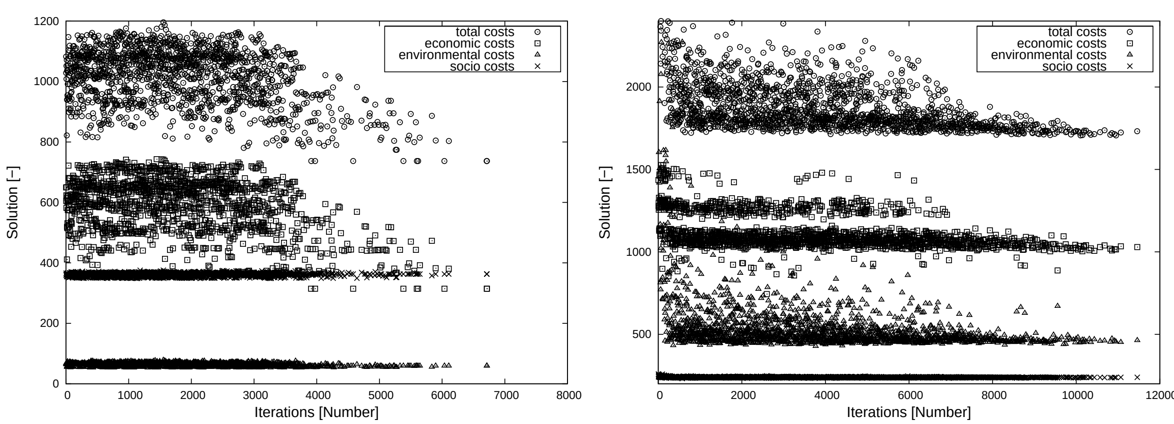


Figure 4.16.: Case Study $P2$: Progress of the optimisation on individual criteria for the case studies *P1* and *P2*

Model Efficiency - For the purpose of testing the developed model's efficiency, it is applied on the expanded study of Vink and Schot (2002) with 48 interconnected wells. The authors use hypothetical ecological impact and lumped economic costs functions for each of these wells and leave the capacities of the interconnecting pipes unlimited. The theoretical number of possible production configurations is then defined as $R = S^N$, where N is the number of wells and S is the number of discharge rate steps per well. The number of feasible configurations is constrained by the continuity equation at each node and continuity of flow on network arcs. For a system of 15 wells with 10 discharge rates and 30% of available spare capacity within the network the number of feasible combinations is in the range of 10^{12}.

Vink and Schot (2002) applied the Genetic Algorithm optimization method to solve their hypothetical study and proved that it performs significantly better than the Monte Carlo procedure. Their Genetic Algorithm procedure with a stationary population size of 220 solutions [9] and mixed *arithmetical* and *uniform* crossover technique [10] needed from 10.000 up to 100.000 generations to approach the analytical optima for the problems with 4 to 48 wells. As it can be seen in Figure 4.17 the applied Simulated Annealing algorithm, accommodated for the network optimization problem in a path form, managed to reach the optimal solution in less then 1.000 for the problem with 10 and in approximately 30.000 iterations for the problem with 48 wells. This represents significant improvement in comparison to the Genetic Algorithm. Nevertheless, the applied algorithm is expanded to work with a set of solutions instead with a single one and the total number of function evaluations needed that all solutions reach the same optimum was 15.000 for the problem with 10 and 450.000 for the problem with 48 wells. In contrary to this shortage, the expansion of the algorithm to work with a set of independent solutions helps to better explore the whole solution space and to prove the convergence of the algorithm. Although the detailed results of the Vink and Schot (2002) study were not available and the comparison above is just a rough approximation, the developed model approximately matches the same optimal solution as the Genetic Algorithm method in single-objective and multi-objective optimization.

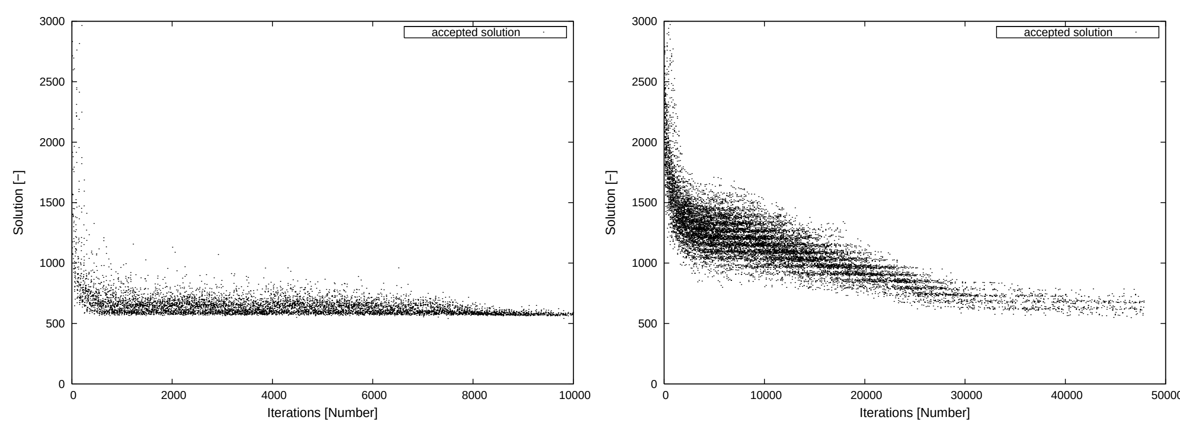


Figure 4.17.: Case Study $P2$: Progress of the optimisation for the case study *P2* with 10 and 48 wells

[9] Genetic Algorithm is inspired by the concept of natural survival of the fittest and is based on biological selection, mutation and inheritance of genetic material among a population

[10] the way of producing of new solutions from already identified ”fittest” ones (the way of combining of genetic information to create new offspring of a population)

4.2. Design Model

Most of the existing water supply systems have been designed and built in the late 19th and the 20th century. As any other man made systems, they are a reflection of the needs, preferences, knowledge level and technical capabilities of the time when they were build up. Although most of these systems are still well functioning, in recent decades, the interests and expectations of water supply decision makers, managers and operators have changed. The importance of better maintenance and operation, public and stakeholder participation, management of water demands and environmental impacts and flexible and reliable systems design and operation are just some of the new driving factors. Instead on focusing only on technical and economic issues, the water supply designers are today increasingly interested in the incorporation of the uncertainty aspects as well as in the risk and reliability issues. A model that supports the development of multi-objective design alternatives, provides for the system uncertainty and reliability quantification as well as risk-oriented system evaluation is presented next.

4.2.1. Characterisation of the Design Problem

In general terms it can be stated that the main purpose of the water supply design is to determine sizes and capacities for some, or all, system components in such a way to provide for the proper functioning of a system under all design conditions for a whole design period (Walski et al., 2003). Since the design conditions are often seen as all stresses which a system is supposed to sustain during its life time, the design of water supply systems components is often achieved by consecutive testing and improving of the system performance for some pre-selected system stresses. Design conditions and design period as well as the main objectives of the design depend on the individual project aims and characteristics, and can differ largely for different systems (e.g. development of a new system or rehabilitation of an already existing one) and the type of a design study (e.g. preliminary design or design of an individual system component). Therefore the main objectives, the level of complexity and expectations from an analysis may also differ greatly. Nevertheless, the most often found objectives in the design of water supply systems can be categorized into:

1. Performance satisfaction, usually in terms of delivered flows and pressures.
2. Costs minimization, usually in terms of investment and operation costs.
3. Benefit maximization, often in terms of reliability of a system.

The first objective is usually considered as a necessary prerequisite for the successful operation of water supply systems and is therefore mainly incorporated as constraint in the design problem formulation, where the performance indicators, such as delivered flows and pressures at demand nodes, have to achieve already established engineering standards such as minimum and maximum node pressures, minimum fire-fighting flows, maximum pipe flow velocities, etc. Although water supply systems are mainly not "market-driven" and many social and institutional factors may predominantly influence their real costs (e.g. subsidies, interests on loans, political interests for infrastructural investments, etc.), the minimization of investment and operation costs is still one of the prime objectives of every design analysis. Costs consist of capital (initial investment) and operation (regular expenditures) part and have to be projected to the same time period mainly using economic the Time Value of Money calculations. The third objective imply the maximisation of the system beneficial value and usefulness to its users. Unfortunately, the benefits of a system are very hard to define and express. Firstly, because each stakeholder (e.g. investors, engineers, environmentalists, consumers) may have different expectations and uses from a water supply project and secondly many benefits such as the contribution to the better health conditions, increase in living standard, rise of the demographic popularity of an area, etc., are extremely difficult to express (Walski et al., 2003). From an engineering point of view the most beneficial are the systems which can perform under a range of different uncertain operating conditions and can sustain a range of possible system failures. This is often seen as the system reliability or the probability of a system not-failure assessment and is here selected as the criteria of the beneficial value of water supply systems.

Decision variables for the design problem are the capacities of system components (e.g. diameters for water pipes and capacities for elements such as treatment plants and pump stations). Since these are directly dependent on flows, flows are selected as the independent variables in the optimization problem. The design of the major part of water supply systems refers to one point in time. This is some high stress condition, such as fire fighting, peak of demand, failure of component, or some combination of the previous. In any case, the decision variables are considered as stationary values. Non-stationarity is important only in the design of the components that transfer water in time (e.g. tanks, reservoirs, etc.) and will be addressed in the next model (operation model). Due to the fact that most of the design variables, such as pipe diameters or pump capacities can be selected only from a discrete set of the available ones at the market, the decision variables are regarded as discrete. As previously stated the constraints for such an optimization problem are the engineering standards in terms of acceptable flow and pressure values as well as the mass and energy conservation equations.

As for the planning problem, the path form of the Minimum Cost Flow Network problem is again used, only this time the Multi-Objective Minimum Cost Network Flow optimisation from the equation 3.43 on page 57 can be reduced to a single-objective one (minimisation of the economic costs) since the performance objective is considered as a constraint and the reliability objective will be considered afterwards. In mathematical terms the optimisation problem is written as:

$$min. \quad z = \sum_{\pi_k \in \Pi} \sum_{a_{ij} \in \pi_k} (DC_{var_{ij}}(x^{\pi_k}) + C_{fix_{ij}}(x^{\pi_k}) y_{ij}) \tag{4.3}$$

subject to:

$$\begin{aligned}
&\sum_{n_j : a_{ij} \in A} x_{ij} - \sum_{n_j : a_{ji} \in A} x_{ji} = b_j && \forall n_j \in N \\
&\sum_{\pi} \delta_{ij}^{\pi} x^{\pi} \leq \kappa_{ij} y_{ij} && \forall \pi \in \Pi \\
&p_{min_j} \leq p_j \leq p_{max_j} && \forall n_j \in N \\
&y_{ij} = 0 \vee 1 && \forall a_{ij} \in A
\end{aligned} \tag{4.4}$$

where x^π is a path flow on a conforming simple path π [11] and the collection of $\mathbf{x} = \{x^{\pi_k} \mid \pi_k \in \Pi\}$ of all conforming paths Π is a network flow vector. Individual arc flows can be obtained as $x_{ij} = \sum_\pi \delta_{ij}^\pi x^\pi$ for $\delta_{ij}^\pi = 1$ if an arc a_{ij} is on the path x^π and 0 otherwise. Unit-functions C are scaled representatives of the net-cost (impact) functions c that depict the impacts of some system parameter such as flow in this case. Furthermore, the variable costs C_{var} are discounted to their net present value DC_{var} in order to bring them to the same time scale as the fixed costs C_{fix}. Already existing system elements have only variable costs ($y_{ij} = 0$), while the potential elements (new or elements under rehabilitation) may have a fixed part ($y_{ij} = 1$), too. Parameter κ_{ij} stands for the upper capacity limit of an arc a_{ij} while p_{min_j} and p_{max_j} stand for the minimum and maximum standard pressure values at a node n_j, respectively.

Although in the design problem a single-objective mathematical formulation of the optimisation problem is used (minimisation of economic costs), the multi-objective nature of the design is encompassed by introducing the performance satisfaction objective as a constraint in the mathematical formulation and by introducing an additional step for dealing with the third objective (reliability maximization or maximization of the probability of not failure). This is necessary, since the probability of a system not failure can be calculated only for an already defined system configuration and represents an additional way to handle complex multi objective problems (decomposition approach). The design problem is separated into: 1) the identification of minimum cost system configurations that satisfy needed performances (**primal solution**) and 2) evaluation of the reliability of these configurations and for different levels of decision makers' risk-tolerance (**final solution**. The selection of the optimal design solution is than a trade-off among system costs and system reliability.

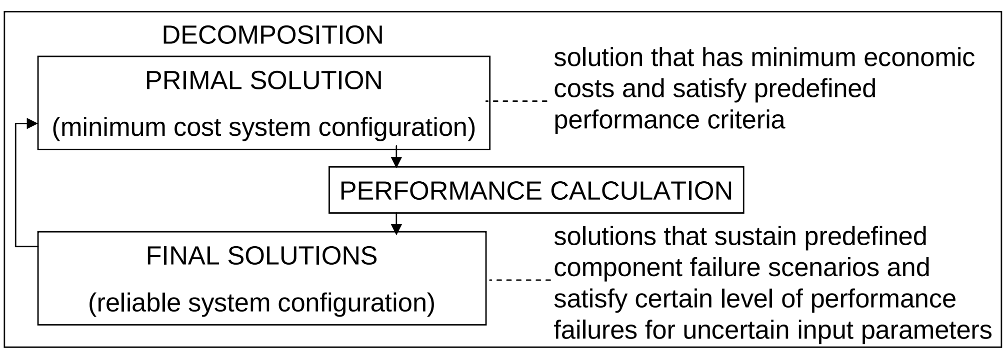


Figure 4.18.: Decomposition applied in the design model

[11] directed path from a source node n_s to a destination node n_d

4.2.2. Accommodation of the Solution Methodology

The solution technique for the defined design optimization problem should be capable of efficiently dealing with the following main tasks:

1. Representation of the water supply system structure and function.

2. Creation of minimum cost design alternative configurations.

3. Reliability assessment based on failure analysis and parameters' uncertainty.

The selected network representation is not just convenient for the water supply systems structure and function representation, but it also has a capability to include layout considerations in the design analysis. As proved by Goulter (1987) the layout of a system significantly influences not just its investment and operation costs but it also affects the reliability of a system. In addition, network representation may be used to improve, or constraint, the optimization algorithms, since it provide for the effective subdivision of the problem into sub-problems on sub-networks. The adopted design model concept based on the directed network representation of water supply systems, is very similar to the Diba et al. (1995) methodology, only the directed graph algorithms are not used just for the pre and post-processing of the optimization algorithm, but they are internally integrated in the optimization procedure. This decreases the computational demand during the exchange of parameters and enable efficient iterative running of the optimisation procedure. Furthermore, the general procedure for the identification of the minimum cost network flows from Jensen (1980) is combined with the connectivity analysis[12] of Ostfeld and Shamir (1996); Ostfeld (2005), in order to promote the exploration of the entire network topology when developing alternative design options. In addition, the first algorithm is accommodated to deal with the minimum cost flow problem defined in the path form and the consideration of the pressure constraint is added to the second algorithm. Although many optimization models work as well without any particular system representation, it may be stated, that exactly the possibility to clearly represent water supply systems structure and function within the optimisation model may be the prevailing factor in increasing the acceptance and applicability of the optimisation methods.

Since the water supply distribution network design problem itself (selection of the sizes for N elements from a predefined set of M sizes) has an exponential time complexity function $O(M^N)$ and very complex functional relations among criteria and system parameters (e.g. flow and pressure distribution depend on the whole network configuration), large water supply systems are often too complex to be solved by exact (analytical) optimization methods (Walski et al., 2003). The methods that create a possible solution, or a set of solutions, check the function value against already obtained solutions and iteratively progress toward more optimal solutions are often referred as approximate methods and present a good alternative for exact methods. Although they do not guarantee the identification of the global optimum and declare only the best found solution, they are often able to identify not just one but a set of very good (near optimal) solutions. The optimization procedure suggested

[12] a water supply systems layout analysis based on the examination of the paths between all individual source and demand nodes

here is composed of the Simulated Annealing algorithm, that solves the minimum cost flow network problem defined in the path form, and the Branch and Bound method, that control the creation and evaluation of all feasible system configurations.

Finally the capability of handling of two main types of failures (component and performance failure) is of crucial importance for the design method that aims to address the system reliability issue. The reliability (expressed as the probability of not failure) is incorporated into a system either by designing for a deterministically determined "worst-case" scenario or by designing with the "uncertainly" defined system parameters. Although the first approach is an elementary part of all standard textbooks on water supply design, it has been judged that it designs systems for a conditions which may newer occur and which in turn often results in the over-dimensioned systems (Tillman et al., 1999). The main difficulty of the second approach is the quantification of uncertainties. Although deterministic, probabilistic, stochastic and entropy based approaches have already been tried, quantification of the parameter's uncertainties in water supply systems proved to be a very hard task (Lansey, 2000). Instead of selecting among one of these two approaches, their combination is suggested. The deterministic or traditional approach is suggested for the component failure analysis, since such scenarios can be easily deterministically defined, and the stochastic approach is suggested for the analysis of the system performance with uncertain input parameters. Only, instead of trying to design a system that can accommodate for the given uncertainties, the backward going approach is used. The alternative system configurations are first produced and their performance for the pre-defined parameter's uncertainty are then calculated. The statistical evaluation of the calculated performance is used to obtain the measure of the system reliability that is considered as a surrogate measure of the quality of a system.

For each individual component failure scenario, the water supply system under consideration is upgraded in order to be able to sustain it with its full performance. An advanced Path Restoration Method of Iraschko et al. (1998) and Iraschko and Grover (2000) is employed to identify the minimum cost network capacity increase that provide for globally optimal network configuration. The parameter's uncertainties are probabilistically defined and divided into different uncertainty levels, here named "threshold". These levels corresponds to the different risk perception levels (e.g. one can choose the 10 % variation as enough buffer capacity for the uncertainty in water demands while someone else may promt for 30 %). The Latin Hypercube Sampling technique is used to produce the samples that are within the "threshold" range and fit to the defined parameter probability density function. These samples are applied on the selected system configurations and their reliability is assessed by statistically evaluating the obtained performance indicators (flows and pressures calculated by a network solver). The cost increase for each component failure scenario and the reliability measure for each offered system configuration for different levels of risk perception are recorded and serve as a basis for the trade off among costs and reliability according to some predefined decision maker's level of risk acceptance.

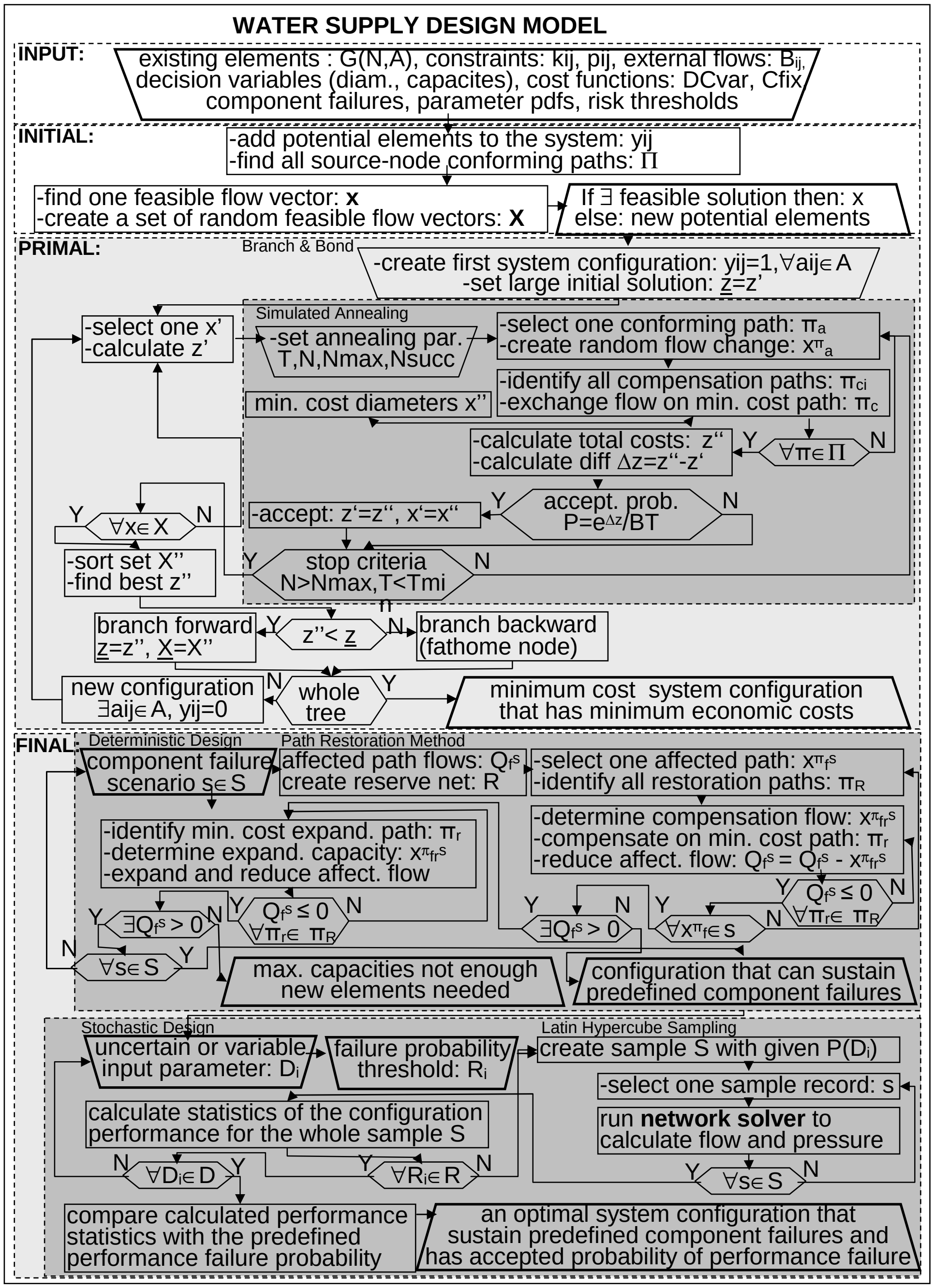


Figure 4.19.: Flow chart of the design model

The solution procedure presented at the previous Figure (Figure 4.19) consists of the following main steps:

1. *Input* - Beside basic water supply network data, such as existing configuration, maximum available water amount at sources, predicted consumer demands, maximum capacities of the transport facilities and pipe connections as well as their hydraulic properties, the characteristics of the potential elements have to be provided. These are their

potential position and set of discrete values of their possible capacities together with investment and operation costs functions. In addition, the component failure scenarios, the probability density function of the uncertain parameters and the acceptable risk "threshold" values for the reliability evaluation have to be defined.

2. *Initial solution - feasible solution without costs* - A graph procedure based on the allocation of maximum flows on paths between source and demand nodes is employed to identify one flow vector that satisfy all demands and does not violate capacity constraints. Cost functions are not considered for this solution.

3. *Primal solutions - minimization of costs* - The Branch and Bound algorithm is used to explore all possible system configurations (addition of potential elements) while the Simulated Annealing algorithm is employed to identify the minimum cost network flow for which the minimum cost pipe diameters are determined. Different system configurations are compared until all branches of the Branch and Bound tree are explored. The primal solution essentially represents the minimum cost water supply system configuration in terms of its layout and component's capacities.

4. *Final solutions - maximization of reliability* - In order to increase the reliability of the identified configuration, the predefined component failure scenarios are incrementally ran. For each scenario, minimum cost spare capacities are added to the system in a way to provide its function without the failed component. The degradations of the minimum cost objective for each failure scenario are recorded. Additionally for each risk-acceptance "threshold" value, the system reliability is assessed by statistically evaluating the network flows and pressures calculated with network solver of Gessler et al. (1985), for the samples of uncertain parameters (e.g. water demands) created with an advance sampling method of Iman and Shortencarier (1984). Since the "threshold" values correspond to the uncertainty levels that one has to accept as the range of possible deviations of the uncertain parameters, they define the acceptable risk level that the decision makers are ready to accept in selection of the solution. The risk acceptable level, system cost and its reliability represent the main criteria for the selection of the final design solution.

4.2.3. Case Study $D1$ - Design Model Demonstration

Study Description - In order to present the purpose and illustrate some capabilities of the developed design model, the same case study as for the planning model is used. This is an adaptation of the study of Alperovits and Shamir (1977) that considers the design of a water distribution network with 4 water sources (river, groundwater and two spring water sources), 6 consumer nodes (out of which 2 are new) and 11 arcs that connect these elements. In addition to the network description and characteristics given in subchapter 4.1.3 on page 72, the set of commercially available water pipes and their costs per unit meter of length had to be defined. The set of 14 pipe diameters, that is mainly used in water supply optimisation literature, where pipe diameters are given in *inches* (1 *inch* = 25.4 *mm*) and pipe investment costs (fixed costs) are given in dollars per meter of length, is selected as the set of the possible decision variables. It originate from the same study of Alperovits and Shamir (1977) and is presented in Table 4.4. It is to be noticed that the operation costs (variable costs) are not included in the standard formulation of the water supply design problem and will be addressed in the next management stage, namely in the operation stage.

D [inch]	1	2	3	4	6	8	10	12	14	16	18	20	22	24
D [mm]	25.4	50.8	76.2	101.6	152.4	203.2	254.0	304.8	355.6	406.4	457.2	508.7	558.8	609.6
C [$/m]	2	5	8	11	16	23	32	50	60	90	130	170	300	550

Table 4.4.: Case Study $D1$: Standard set of available pipe diameters with their investment costs per unit length [source: Alperovits and Shamir (1977)]

Problem Statement - The design analysis is logical extension of the planning analysis in which for some identified network general configuration the capacities of network elements are to be determined. Therefore one of the Pareto-optimal planning solutions, presented in the previous chapter (Environmentally optimal) is used as an input network configuration for the design analysis. The selected solution is the one that favours the use of all three new water sources (*N8, N9, and N10*) and suggests building of the new transport arcs *A8, A9, A10, A11*. In addition to these new elements, the rehabilitation of the existing arcs *A4* and *A6* is also included in the consideration. The network configuration and the characteristics of the selected planning solution are given in Figure 4.20.

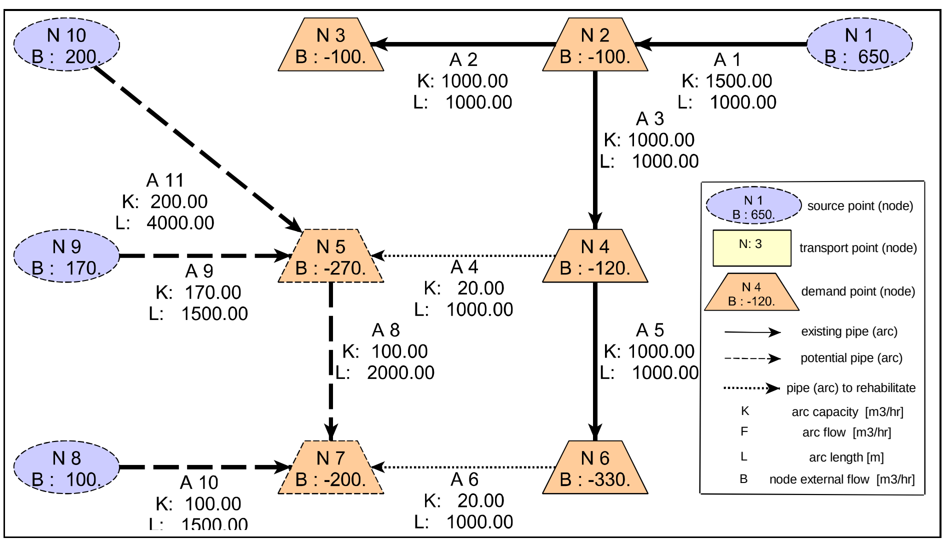


Figure 4.20.: Case study $D1$: Network configuration of the selected planning solution

The stated objectives of the performance satisfaction, costs minimisation and benefits maximisation are obviously conflicting. The smallest possible elements sizes that provide for the satisfactory flows and pressures, within some water supply network yield the minimum investment costs. The reliable functioning of a water supply system under different operating conditions, uncertain parameter values and emergency or failure situations, demands for the existence of some spare capacities, whose addition obviously ruin the minimum investment cost criterion. The compromise among these two objectives is the predominate question in the design of water supply systems. Due to the fact that the reliability assessment can be done only for already defined systems, a two step approach is adopted for the integration of economic and reliability objectives. The first is the minimisation of the investment costs, while second is further divided into the reliability increase for the preselected failure scenarios and the reliability assessment of the systems' performance for some predefined levels of parameters' uncertainty.

Primal Solution - Minimum Cost Solution - Same as for the planning model, the maximum feasible flow network algorithm of Jensen (1980) is used to identify first feasible solution. By changing flows on conforming paths for each source-destination node combination, this feasible solution enables for the creation of new random but feasible solutions and serves as the beginning point for the rest of the optimisation procedure. The combination of the Branch and Bound and the Simulated Annealing algorithm is used to identify the flow vector for which the investment costs of the pipe diameters are minimal (primal solution). Since the design problem is mathematically defined as a single-objective one (Equation 4.3) the optimization procedure considers only economic costs. It basically, explores different combinations of system configurations with the Branch and Bound algorithm, identifies minimum cost flow solution for each configuration with the Simulated Annealing algorithm and calculates minimum cost pipe diameters for these flows. Last two steps are repeated until the minimum cost solution out of all possible configurations is identified. Calculated network flows and pipe diameters of the primal solution are presented in Figure 4.21.

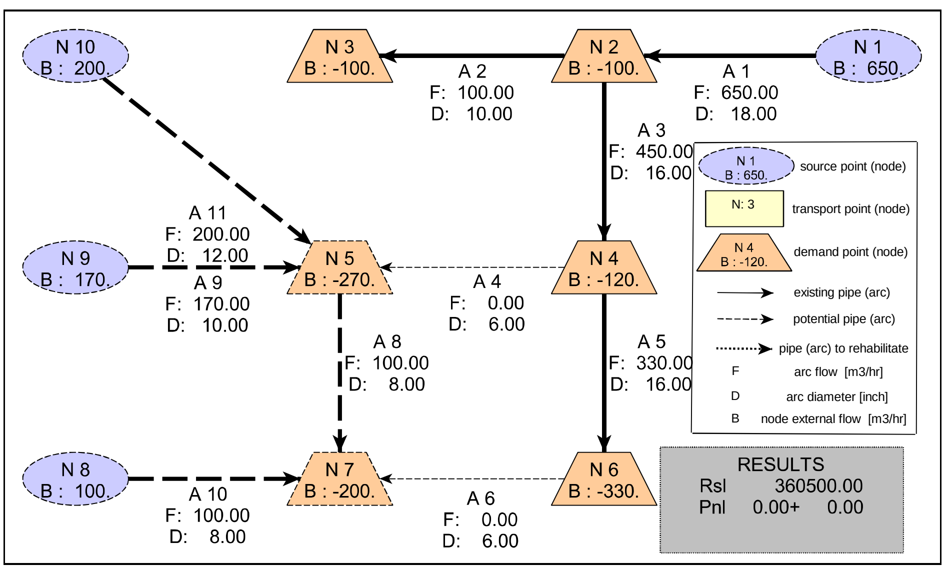


Figure 4.21.: Case Study $D1$: Identified primal solution

The primal solution is the minimum investment cost solution that provides for the satisfaction of flow and pressure constraints. The diameters identified for the new pipes ($A8, A9, A10, A11$) are the minimum diameters that provide for the delivery of the demand flows and the satisfaction of the minimum pressure of 30 m at each node. The calculated flows and head losses in arcs as well as the delivered pressures at nodes are presented in Table 4.5. Similar as for the planning problem, the primal solution will be used as the reference one, only now, it is not expected to achieve further improvements on the economic criteria but instead by increasing the reliability of the system an increase in costs is expected.

Arc ID	Length	Friction coefficient	Flow	Head loss	Diameter	Diameter
org.	[m]	C	[m3/day]	[m]	[inch]	[m]
1	1000	130	650.00	2.46	18	457.2
2	1000	130	100.00	1.35	10	254.0
3	1000	130	450.00	2.21	16	406.4
4	1000	130	0.00	0.00	6	152.4
5	1000	130	330.00	1.25	16	406.4
6	1000	130	0.00	0.00	6	152.4
8	2000	130	100.00	7.99	8	203.2
9	1500	130	170.00	5.40	10	254.0
10	1500	130	100.00	5.99	8	203.2
11	4000	130	200.00	8.00	12	304.8

Node ID	Elevation	Head	Pressure
org.	[m.a.s.l.]	[m.a.s.l.]	[m]
1	150	180.00	30.00
2	120	177.54	57.54
3	130	176.19	46.19
4	125	175.33	50.33
5	120	175.33	55.33
6	135	174.08	39.08
7	130	167.34	37.34
8	150	180.00	30.00
9	150	180.00	30.00
10	150	180.00	30.00

Table 4.5.: Case study $D1$: Calculated flow, head loss and pressures for the primal solution

It is to be noticed that the general network procedure for the solution of the Minimum Cost Flow Network problem of Jensen (1980) has been accommodated in order to include the pressure distribution over a water supply network. The Simulated Annealing algorithm is based on randomly generated flow changes on network paths and for each flow change, a small inner algorithm for the determination of pipe diameters, such that the pressure conditions downstream of this pipe are satisfied, is employed to determine the feasibility of this flow change. This enables to directly determine minimum cost pipe diameters for any created flow change. It is to notice, that for a given flow and pressure conditions, the determination of the pipe diameters is a trivial problem only for one path network (linear network from one source to one demand). On semi-looped and looped networks a change of one pipe diameter affects the pressures on all downstream nodes. For this problem the definition of the flow vector on simple conforming paths turned to be extremely useful and enabled ease identification of all affected nodes and determination of the minimum pressure conditions at the end of a pipe under investigation as the minimum pressure form all downstream paths. In order to avoid iterative determination of the diameters along one path, the diameters are determined by investigating arc in upstream order to the direction of the flow along a path under consideration.

Final Solution - Component Failures - The minimum cost design solutions are used in the literature to state the efficiency of some optimisation procedure but are of very little practical value. Water supply systems have to operate not only for the design conditions but also have to be able to sustain a wide range of stress conditions that may occur during their life period. The failure of some network component is one of the most common stresses and any practically oriented design has to be able to address this issue. The identification and selection of the components that are prone to failure is another important issue but it is very system-specific and can not be easily generalised into one methodology applicable for various systems. The attention of this study is on the development of the method that enable systematic and minimum-costs upgrades of the system capacities for some predefined component failure scenarios. The method suggested is based on an advanced path restoration method of Iraschko and Grover (2000) that produces the minimum cost network spare capacity additions by reconfiguring the flow paths on a whole network. It also had to be accommodated to consider the pressure constraints in the selection of the possible paths that can compensate for some individual failure.

For the implemented case study, the failures of all arcs that supply water to the demand nodes *N5* and *N7* are considered. These are namely arcs: *A8, A9, A10, A11*. It is to be noted, that the restoration algorithm considers 4 new arcs (*A8, A9, A10, A11*) as well as 2 existing arc that can be rehabilitated (*A4, A6*) as eligible for the addition of the spare capacities. The results of the component failures analysis for arcs *A8, A9, A10, A11* and resulting increase in pipe diameters of the network are presented in Figure 4.22.

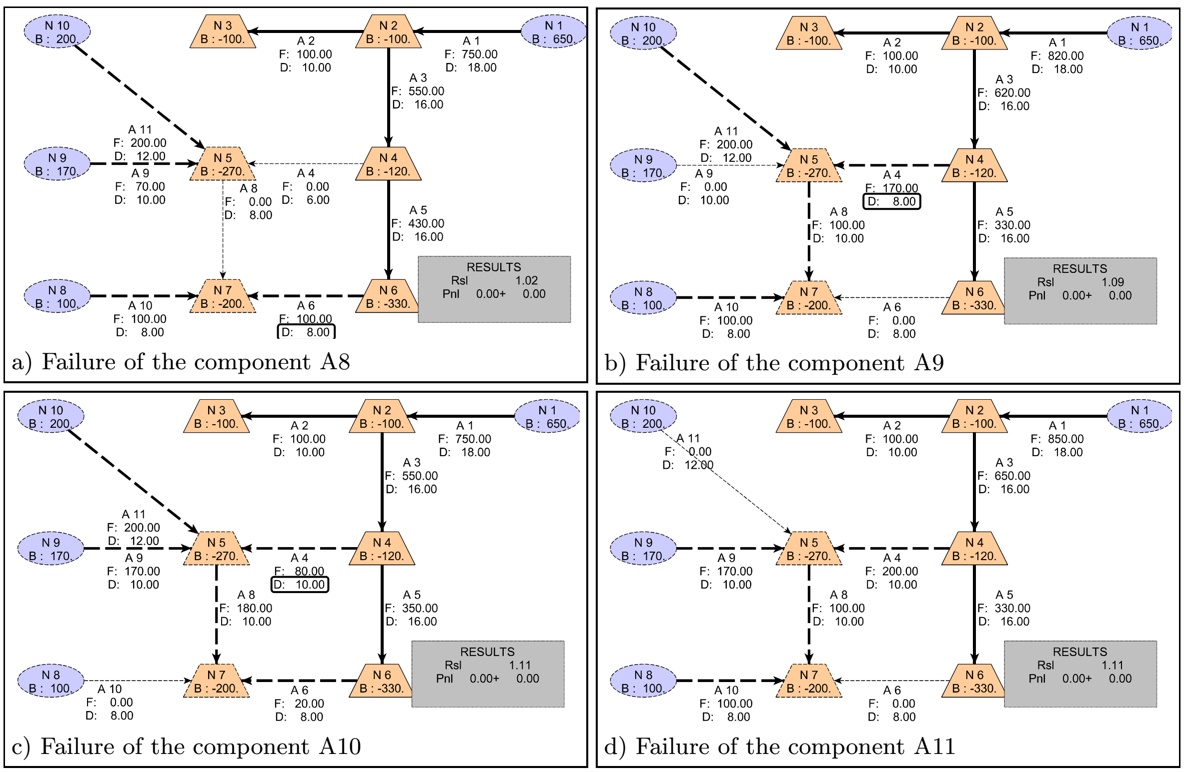


Figure 4.22.: Case study $D1$: Increase of the network capacities for selected component failure scenarios

Since the adding of spare capacities is a minimum cost oriented optimization, for the failure of the component *A8* the algorithm identifies the capacity increase of the existing arc *A6* from 6 *inch* to 8 *inch* as the minimum cost option (graph *a* in Figure 4.22). This capacity increase enables transport of necessary 100 m^3/day through the arc *A6* with the encountered costs increase of only 2 % to the total system costs. Similarly, for the failure of the arc *A9* the algorithm identifies the expansion of the capacity on the existing arc *A4* from 6 to 10 *inch* as the minimum cost option with encountered costs increase of 9 % to the total costs (graph *b* in Figure 4.22). Both these options are more then obvious since the rehabilitation of an existing arc is defined as cheaper option than the building of a new one. By further evaluation of the failures of the components *A9* and *A10* the previous upgrades are remembered and the optimisation procedure identifies the increase of the diameter on the new arc *A8* from 8 to 10 *inch* and on the existing arc *A4* from 8 to 10 *inch* as the ones that provide enough spare capacities for the compensation of the failed flows. The biggest advantage of such an approach is that it provides for the identification of the minimum cost network paths that use both existing and new capacities to their full capacity in order to satisfy for some predefined failures of individual components.

The increase in costs by provision of the additional capacities on arcs *A4,A6* and *A8* that provide for the functioning of the system in case of failures of the arcs *A8,A9,A10* and *A11* is shown in Figure 4.23. It can be seen that the total increase in costs of 11,4% provides for the compensation of all defined failure scenarios. In addition the offered solutions still satisfy all constraints since the penalties on arcs and nodes are equal zero. Furthermore, the artificialy introduced "penalties on slacks"[13] are also equal zero. This means that the cpacity restoration algorithm has managed to identify at least one feasible sloution for each failure scenario and that the initially provided maximum network cpacities are enough for the expansion according to the predefined components failures.

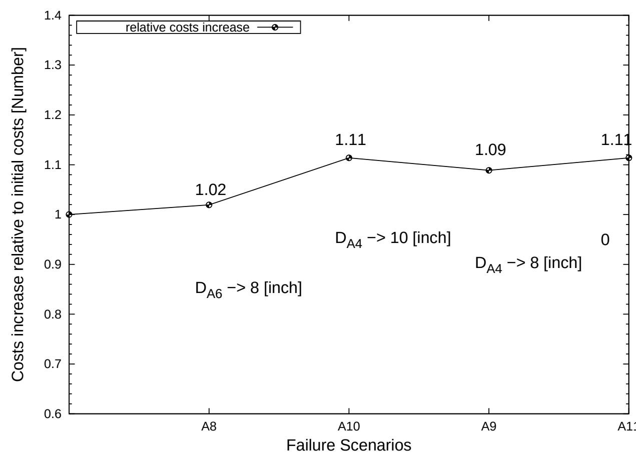


Figure 4.23.: Case study $D1$: Relative increase in investment costs for selected component failure scenarios

Final Solution - Performance Failures - In addition to the variable operating conditions, the design analysis has to address the question of the variable and uncertain design parameters (e.g. water demands, water supply, hydraulic characteristics of the system, etc.). Since these parameters are predicted input values, whose accuracy can be proved only during later phases of the system exploitation, their variability and uncertainty have to be incorporated into the design. Instead of trying to design systems that cover for all occurrences of uncertain parameters, the approach that evaluates the potential (probability) of some suggested system configuration to sustain for some probabilistically defined uncertaint parameters is adopted. This potential essentially define system reliability and is calculated as the statistical evaluation of the system behaviour for samples of uncertain variables. This statistics is than the basis for the acceptance or identification of the need for further improvement of some solution based on the risk acceptability of a decision maker.

For the illustration of the methodology, water demand at all demand nodes (*N2,N,3,N4,N5,N6,N7*) of the Alperovits and Shamir (1977) problem is considered as an

[13] punischment value on virtual arcs that connect nodes of the network with one virtual node (slack node) and provide for the balancing of external flows

uncertain variable. The uniform probability density function is adopted and the reliability of the system is evaluated for two uncertainty levels (15 and 30 %). It is adopted that these "threshold" values define also the acceptable risk level of some decision maker. The Latin Hypercube Sampling method is used to create two samples of 29 points that represent the distribution of the occurences of the uniformly distributied uncertain demands at all six demand nodes with 15 and 30 % uncertainty levels (Figure 4.24).

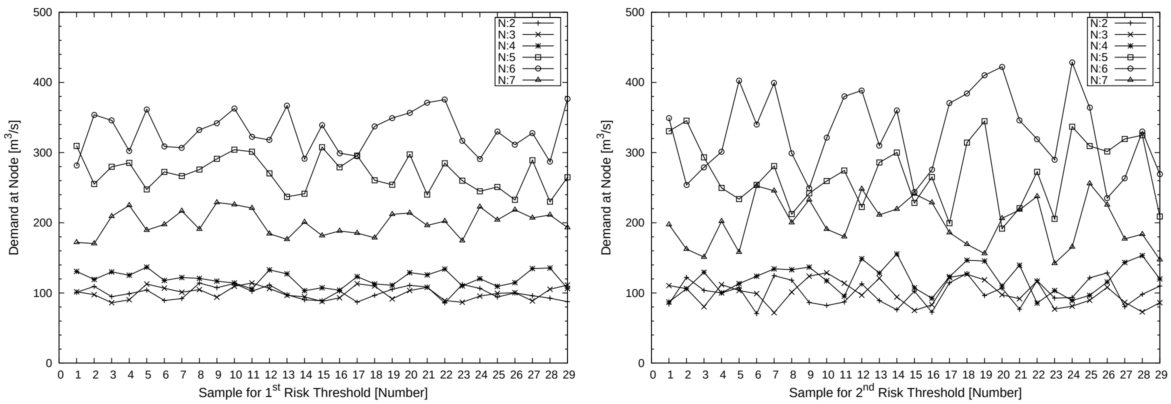


Figure 4.24.: Case study $D1$: Independent and uniform water demand samples with 15 % and 30 % uncertainty tresholds

Figure 4.25 shows the statistics of the above plotted samples. It is to notice that demand variations are uniformly distributed at each node and that the amplitude of the deviations corresponds to the magnitude of the demand at each node. This proves the ability of the Improved Latin Hypercube Sampling technique to create samples according to some predefined probability density function and with a given magnitude of deviations.

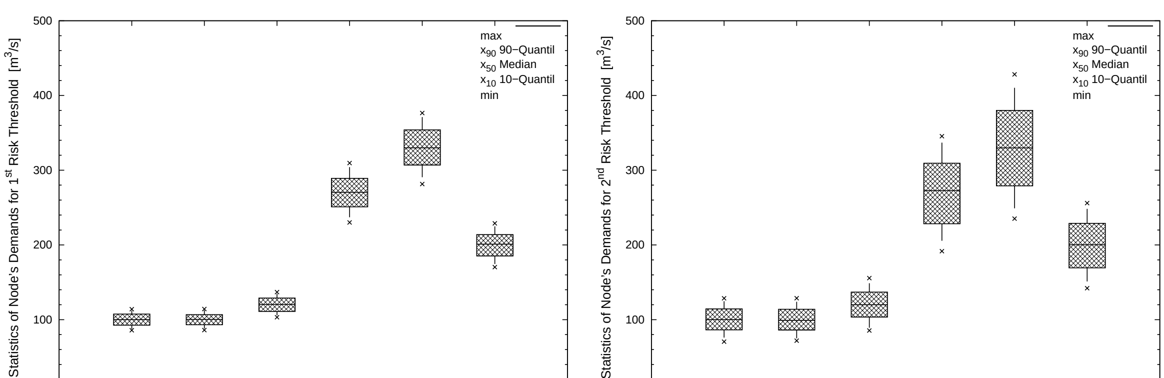


Figure 4.25.: Case study $D1$: Statistic of the water demand samples with 15 % and 30 % uncertainty tresholds

The network solver of Gessler et al. (1985) based on the network solution method of Gessler (1981) is used to calculate the flows and pressures for both samples. Produced results are statistically evaluated for each arc and node in terms of flow and pressure statistics. The number of points in the samples (i.e. 29) is accidental but in essence should be selected in a way to provide for the reliable calculation of the flow and pressure statistics. Looking at the arc flows (Figure 4.26) and the pressures distribution within a network (Figure 4.27) can be concluded that both parameters stays within the predefined constraints ($x_{ij} \leq \kappa_{ij} y_{ij} \forall a_{ij} \in A$ and $p_{min_j} = 35m \leq p_j \leq p_{max_j} = 65m \forall n_j \in N$). Furthermore even if the deviations within the second sample are much greater then in the first one, due to the inherent equalisation and redistribution of flows and pressures within branched networks, the deviations in the obtained flows and pressures are quite moderate.

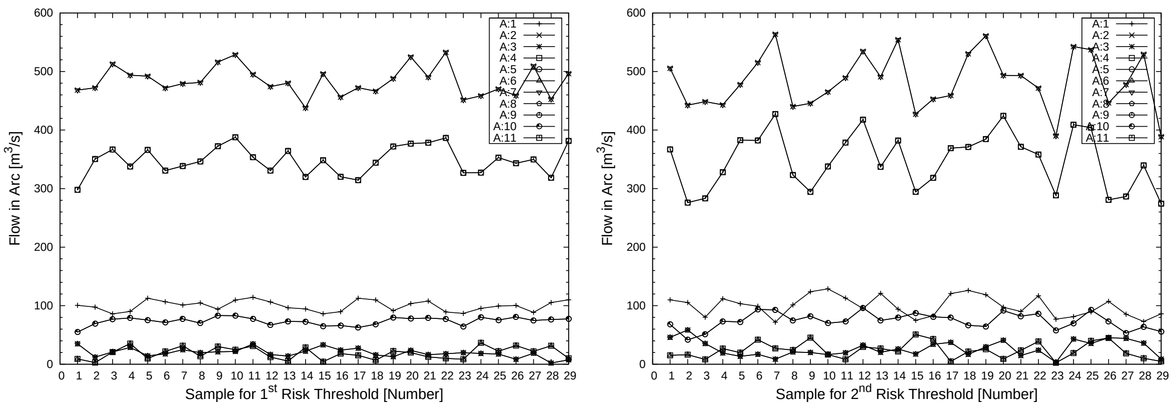


Figure 4.26.: Case study $D1$: Obtained flows in arcs for demand samples with 15 % and 30 % uncertainty tresholds

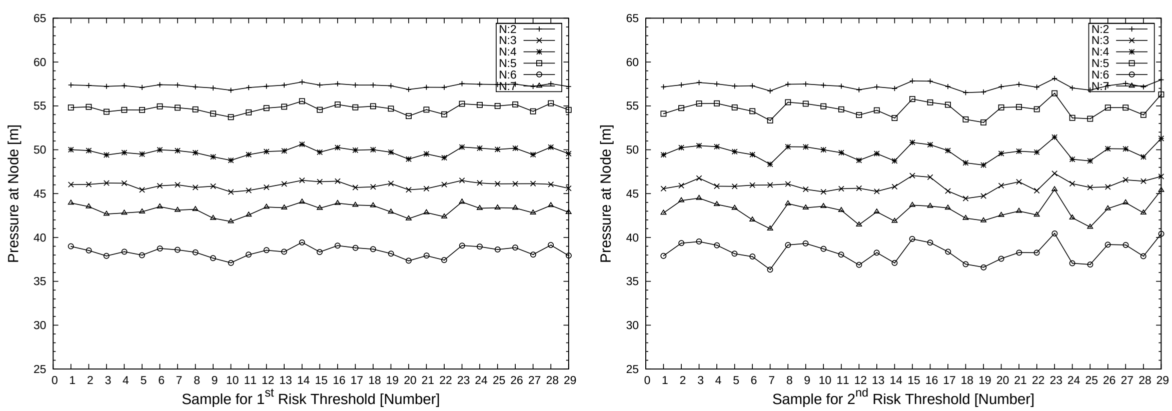


Figure 4.27.: Case study $D1$: Obtained pressures at nodes for demand samples with 15 % and 30 % uncertainty tresholds

Looking at the statistic of the arc flows (Figure 4.28) and nodal pressures (Figure 4.29) the same conclusion can be obtained. Such good performance of the suggested network is generally due to the implemented spare capacities during the component failure analysis. Even the node with a very low pressure (*N6*) has minimum occurred pressure value above limit of 35 m. A good performance of the network means high reliability level. For the here adopted demand's uncertainty of 15 % and 30 % from the predicted values with uniform probability density function and no independence of individual node water demands a reliability of 100 % in terms of network flows and pressures is obtained.

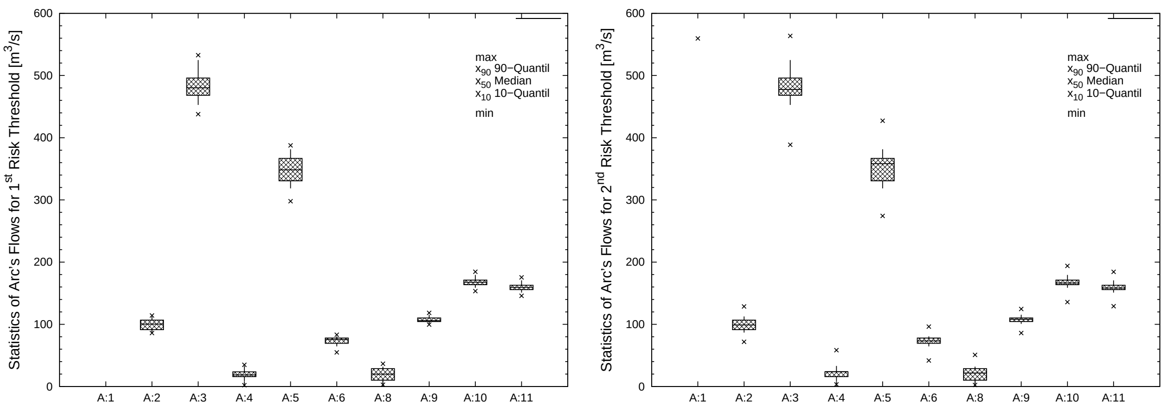


Figure 4.28.: Case study $D1$: Statistics of the calculated arc flows

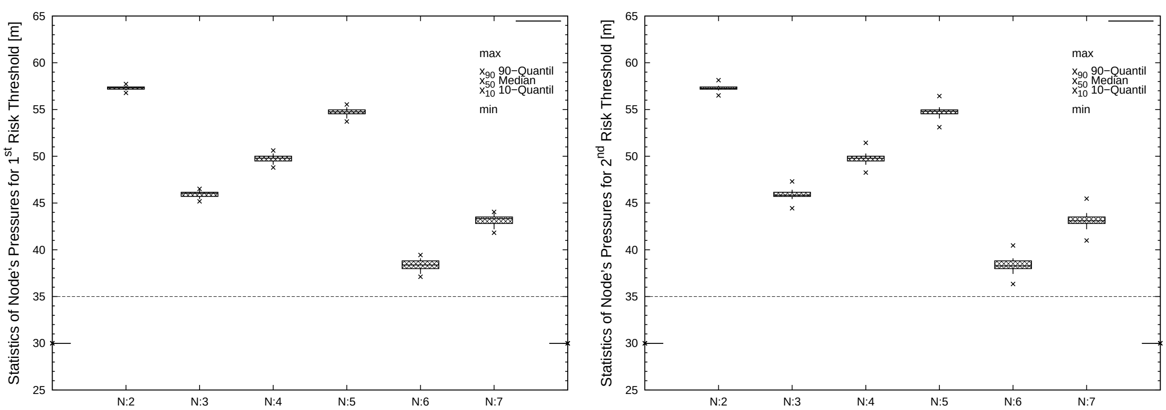


Figure 4.29.: Case study $D1$: Statistics of the calculated nodal pressures

It is to noticed that for the creation of the above samples (Figure 4.24) the independence among water demands has been assumed. Since in reality it often occurs that the demands, or some other uncertain variable such as system friction coefficients, are mutually dependent or at least share similar trends, the adaptation of the Latin Hypercube Sampling method of Iman and Conover (1982) for inducing rank correlation among input variable has been implemented. For the illustration purposes a very strong rank correlation among water demand at all 6 nodes is introduced and presented in matrix 4.5.

$$
\begin{array}{c}
\\
N2 \\ N3 \\ N4 \\ N5 \\ N6 \\ N7
\end{array}
\begin{array}{c}
\begin{array}{cccccc} N2 & N3 & N4 & N5 & N6 & N7 \end{array} \\
\begin{pmatrix}
1.0000 & & & & & \\
0.8010 & 1.0000 & & & & \\
0.9532 & 0.6458 & 1.0000 & & & \\
0.9429 & 0.6369 & 0.9961 & 1.0000 & & \\
0.9374 & 0.6005 & 0.9887 & 0.9798 & 1.0000 & \\
0.9167 & 0.5734 & 0.9626 & 0.9473 & 0.9887 & 1.0000
\end{pmatrix}
\end{array}
\tag{4.5}
$$

The now obtained sample for the water demand's uncertainty with a "threshold" value of 30% and the statistics of the calculated nodal pressures are shown in the Figure 4.30. It can be seen that induced rank correlation among input variables cause an evident affect on the performance of the system. The pressure at the demand node *N6* now reach the minimum limit of 35 m. Nevertheless, the statistical evaluation of the pressures at the node *N6* shows that such events lay in the lower 10% quantile of the calculated pressures and have a very low probabilty of occurence. It can be said that the probability of a failure for the adopted mutualy dependent and uniform demand's uncertainties with the treshold value of 30 % is less then 10 %. Based on his own risk perception the decisin maker may now decide whether such performance failure probability is acceptable or not.

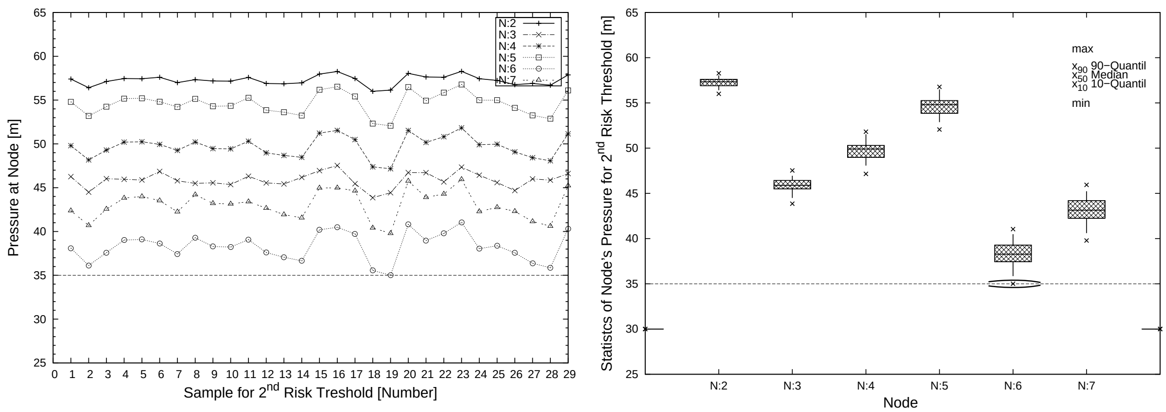


Figure 4.30.: Case study $D1$: Correlated and uniform water demand samples with 30 % unceratinty threshold and corresponding calculated nodal pressure statistics

As just shown, the method applied in this study provides for the completely transparent evaluation of the uncertainty, failures and reliability of water supply systems. This has been seen as a good way to promote greater involvement and participation of the decision makers since they are not just involved in the selection of some predefined alternatives but the alternatives are accommodated to their perception of the needed system performance and reliability. Furthermore, the multi-objectivity of the design problem is implemented too. For example the risk prone decision makers may sacrifice some of the system performances or system reliability for some savings in costs. Nevertheless the consequence of such sacrifices (accepted failures of the system) have to be considered carefully. The failures that cause low pressures in the network have to be distinguished from the ones that cause interruption of continuous water supply. Finally the approach provide for explicit consideration of the parameters' uncertainty and variability during the analysis of the system. This should add to the identification of more robust and flexible development options that may improve the long term management of water supply systems.

4.2.4. Case Study $D2$ - Design Model Validation

For the water supply network design problem some standard case studies that serve for testing of the validity and efficiency of optimisation models exist. Since the design of looped networks is a much complexer combinatorial problem than the design of branched ones, two standard looped case studies are applied here for the validation and efficiency testing of the developed design model. The first one is the already presented 2-loop network of Alperovits and Shamir (1977) in its original form and the second one is the 3-loop network of Fujiwara and Khang (1990).

2-Loop Study Description - In its original form, the 2-loop network of Alperovits and Shamir (1977) has only one supply node (*N1*) that supplies water to 6 demand nodes connected with 8 water pipes. The characteristics of the network arcs and nodes are already given in Table 4.1 on page 73 as well as the set of available pipe diameters with accompanying investment costs that are provided in Table 4.4 on page 98. The configuration of the network itself is presented in Figure 4.31.

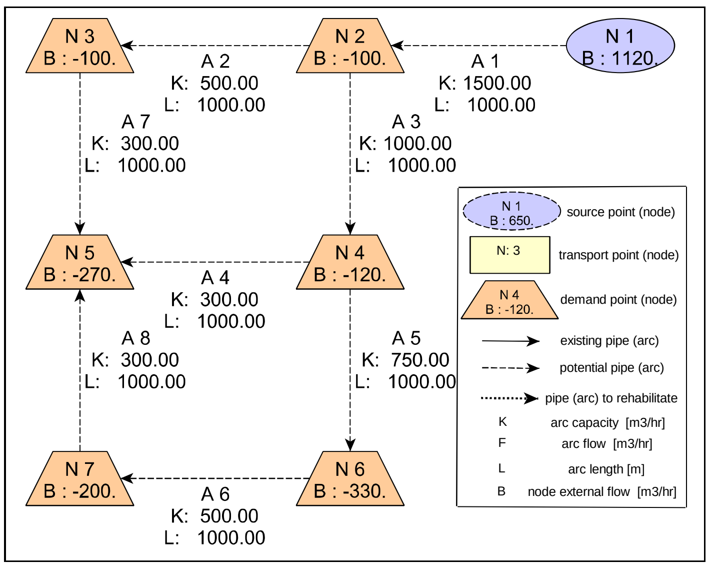


Figure 4.31.: Case study *D2a*: Network configuration of the 2-loop network [Alperovits and Shamir (1977)]

3-Loop Study Description - The Fujiwara and Khang (1990) network also has only one source node (*N1*) that supply water to 31 demand nodes enclosed by a network of 34 pipes. It also serves as an exemplary water distribution network design problem in which the minimum cost pipe diameters are searched for. The characteristics of network arcs and nodes are given in Table 4.6 while the network configuration is presented in Figure 4.32.

Arc ID	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
L [m]	100	1350	900	1150	1450	450	850	850	800	950	1200	3500	800	500	550	2730	1750
Arc ID	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34
L [m]	800	400	2200	1500	500	2650	1230	1300	850	300	750	1500	2000	1600	150	860	950

Node ID	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Q [m^3/s]	19940	-890	-850	-130	-725	-1005	-1350	-550	-525	-525	-500	-560	-940	-615	-280	-310
Node ID	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32
Q [m^3/s]	-865	-1345	-60	-1275	-930	-485	-1045	-820	-170	-900	-370	-290	-360	-360	-105	-805

Table 4.6.: Case study *D2b*: Characteristics of the 3-loop network (adaptation from Fujiwara and Khang (1990))

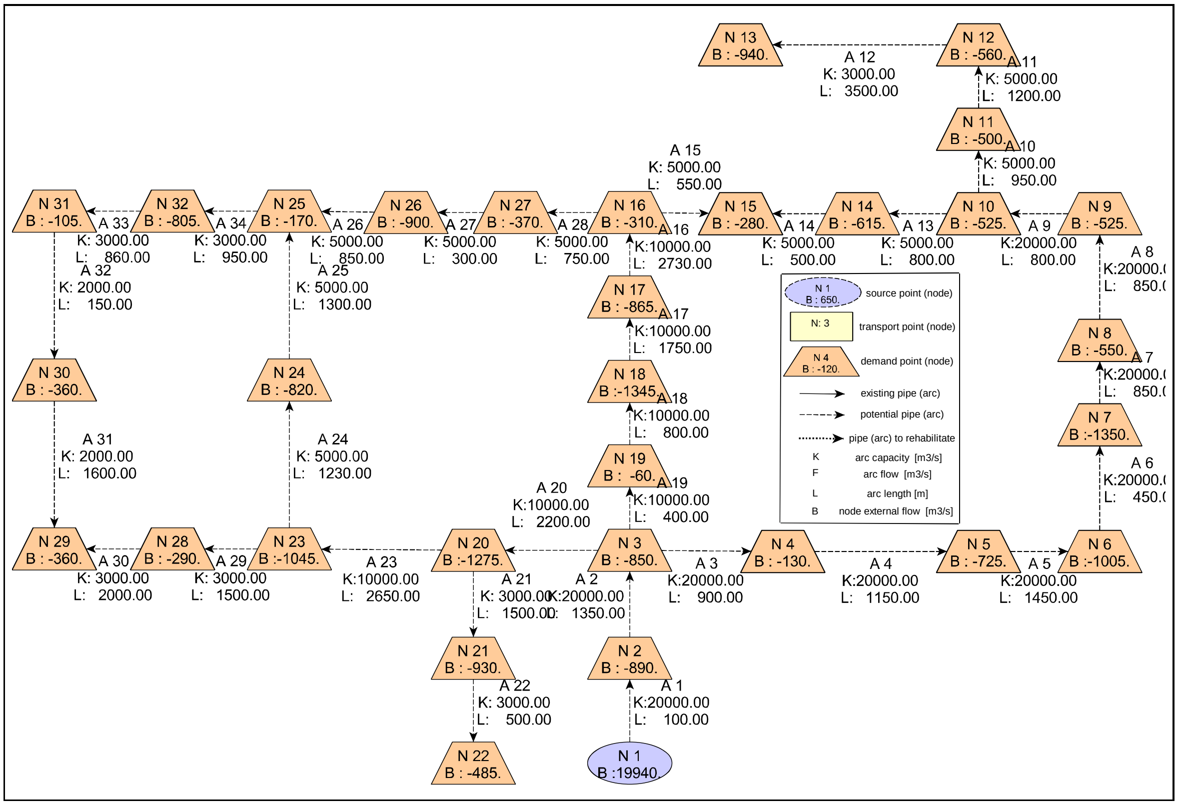


Figure 4.32.: Case study $D2b$: Network configuration of the 3-loop network [Fujiwara and Khang (1990)]

In addition, to the given characteristics, both case studies have to deliver demanded water quantities to the demand nodes and satisfy the minimum pressure head of 30 m and maximum pressure head of 60 $m$at each node. Furthermore, the standard set of commercially available pipes presented in Figure 4.4 on page 98 is for the 3-loop network expanded with 6 additional diameters as in Table 4.7. The investment costs for pipes are defined as a linear function of the pipe length and diameter $C_{fix_{ij}} = 1.1 L_{ij} D_{ij}^{1.5}$ where D_{ij} are pipe diameters in $inch$ and investment costs $C_{fix_{ij}}$ are in $USA\ dollars$ as in the original work of Fujiwara and Khang (1990).

D [inch]	12	16	20	24	30	40
D [mm]	304.8	406.4	508.7	609.6	762.0	1016.0
C [$/m]	45.7	70.4	98.4	129.3	180.7	278.3

Table 4.7.: Case study $D2b$: Additional pipe diameters with their investment costs per unit length [source: Fujiwara and Khang (1990)]

Model Validation - The results obtained with the developed design optimisation method are compared with the Genetic Algorithm method of Savić and Walters (1997), the combination of several search algorithms of Abebe and Solomatine (1998), the Simulated Annealing method of Cunha and Sousa (1999) that has no explicit network representation, the Shuffled

Frog Leaping Algorithm[14] of Eusuff and Lansey (2003) and the Shuffled Complex Evolution algorithm[15] of Liong and Atiquzzaman (2004). More details about these methods can be found in the referred articles and their results will be used here only for the validation of the developed model and testing of its efficiency (as in Liong and Atiquzzaman, 2004).

Arc ID	Savic & Walters (1997)		Abebe & Solomatine	Cunha & Sousa	Eusuff & Lansey	Liong & Atiquzzaman	Comb.BB&SA (this work)
org.	Diam.[inch]		Diam.[inch]	Diam.[inch]	Diam.[inch]	Diam.[inch]	Diam.[inch]
1	18	20	18	18	18	18	**18**
2	10	10	10	10	10	10	**10**
3	16	16	16	16	16	16	**16**
4	4	1	4	4	4	4	**4**
5	16	14	16	16	16	16	**16**
6	10	10	10	10	10	10	**10**
7	10	10	10	10	10	10	**10**
8	1	1	1	1	1	1	**1**
Cost [$]	419,000	419,000	419,000	419,000	419,000	419,000	**419,000**
N.of Eval.	65,000	65,000	1,373	25,000	11,232	1,091	**1,600**

Table 4.8.: Case study $D2a$: Comparison of the obtained solution with in literature reported solutions for the 2-loop network

Table 4.8 presents the minimum cost identified diameters for the 2-loop network of Alperovits and Shamir (1977). Since the developed model (last column in Table 4.8) identify the same minimum costs combination of pipe diameters as all other models it can be stated that it is valid for this case study. This is still not a prove of the general validity and applicability of the model. Nevertheless since such prove can not be theoretically derived for the approximation methods, the validity of the model for some test case studies is considered as an indirect indication of its general validity and applicability. Furthermore, although this problem is not a very complex one (for adopted 14 possible pipe diameters and the network of 8 pipes, the number of capacity unlimited combinations is $14^8 = 1.4 * 10^9$) the identification of the exactly same result ($Cost = 419,000$ \$) by all presented models is the indication of the global optimality of the solution. Nevertheless the difference can be noticed in the number of function evaluations (*N. of Eval.*)that individual models need in order to reach the optimum. As it can be seen in the last row in Table 4.8, the proposed method needs approximately similar number of function evaluations as the, so far best reported, methods of Liong and Atiquzzaman (2004) and Abebe and Solomatine (1998). Still, it is important to keep in mind that the efficiency of each method depends on its parameters that have to be accommodated for each specific optimization. Therefore the presented comparison has only relative value.

Table 4.9 presents the identified minimum cost diameters and corresponding node pressures for the 3-loop case study of Fujiwara and Khang (1990). The number of capacity unlimited combinations for 6 adopted possible pipe diameters on network of 34 pipes is $6^{34} = 2.8 * 10^{26}$. For such complex combinatorial problem it is not surprising that many similar solutions (near optimal solutions) may be found and that the global optimality of the solution is hard to be proved. The considered methods yield different result values. Nevertheless their validity

[14] optimisation technique based on memetic frog transformation and information exchange among the population

[15] optimisation technique based on sorting and subdividing of population into sub-complexes that can evolve independently but are combined to obtain the fittest offspring

Arc ID	Savic & Walters (1997)		Abebe & Solomatine (1998)	Cunha & Sousa (1999)	Liong & Atiquzzaman (2004)	Combination of BB & SA (this work)
org.	D.[inch]		D.[inch]	D.[inch]	D.[inch]	D.[inch]
1	40	40	40	40	40	**40**
2	40	40	40	40	40	**40**
3	40	40	40	40	40	**40**
4	40	40	40	40	40	**40**
5	40	40	30	40	40	**40**
6	40	40	40	40	40	**40**
7	40	40	40	40	40	**40**
8	40	40	30	40	30	**30**
9	40	30	30	40	30	**30**
10	30	30	30	30	30	**30**
11	24	30	30	24	30	**30**
12	24	24	30	24	24	**24**
13	20	16	16	20	16	**16**
14	16	16	24	16	12	**12**
15	12	12	30	12	12	**12**
16	12	16	30	12	24	**30**
17	16	20	30	16	30	**30**
18	20	24	40	20	30	**30**
19	20	24	40	20	30	**30**
20	40	40	40	40	40	**40**
21	20	20	20	20	20	**20**
22	12	12	20	12	12	**12**
23	40	40	30	40	30	**30**
24	30	30	16	30	30	**24**
25	30	30	20	30	24	**20**
26	20	20	12	20	12	**16**
27	12	12	24	12	20	**20**
28	12	12	20	12	24	**24**
29	16	16	24	16	16	**16**
30	16	16	30	12	16	**16**
31	12	12	30	12	12	**12**
32	12	12	30	16	16	**12**
33	16	16	30	16	20	**20**
34	20	20	12	24	24	**24**
Cost [M.$]	6,073	6,195	7,000	6,056	6,220	**6,270**
N.of Eval.	-	-	16,910	53,000	25,402	**19,000**

Node ID	Savic and Walters (1997)		Abebe & Solomatine (1998)	Cunha & Sousa (1999)	Liong & Atiquzzaman (2004)	Combination of BB & SA (this work)
org.	D.[inch]		D.[inch]	D.[inch]	D.[inch]	D.[inch]
1	100	100	100	100	100	**100**
2	97.14	97.14	97.14	97.14	97.14	**97.14**
3	61.63	61.63	61.67	61.63	61.67	**61.67**
4	56.83	57.26	58.59	56.82	57.54	**57.63**
5	50.89	51.86	54.82	50.86	52.43	**52.64**
6	44.62	46.21	39.45	44.57	47.13	**47.46**
7	43.14	44.91	38.65	43.1	45.92	**46.29**
8	41.38	43.4	37.87	41.33	44.55	**44.97**
9	39.97	42.23	35.65	39.91	40.27	**40.88**
10	38.93	38.79	34.28	38.86	37.24	**38.01**
11	37.37	37.23	32.72	37.3	35.68	**36.45**
12	33.94	36.07	31.56	33.87	34.52	**35.29**
13	29.72*	31.86	30.13	29.66*	30.32	**31.08**
14	35.06	33.19	36.36	34.94	34.08	**35.64**
15	33.07	32.9	37.17	32.88	34.08	**35.85**
16	30.15	33.01	37.63	29.79*	36.13	**39.27**
17	30.24	40.73	48.11	29.95*	48.64	**45.42**
18	43.91	51.13	58.62	43.81	54	**52.44**
19	55.53	58.03	60.64	55.49	59.07	**58.54**
20	50.39	50.63	53.87	50.43	53.62	**54.40**
21	41.03	41.28	44.48	41.07	44.27	**45.11**
22	35.86	36.11	44.05	35.9	39.11	**39.95**
23	44.15	44.61	39.83	44.24	38.79	**41.95**
24	38.84	39.54	30.51	38.5	36.37	**37.13**
25	35.48	36.4	30.5	34.79	33.16	**33.22**
26	31.46	32.93	32.14	30.87	33.44	**34.93**
27	30.03	32.18	32.62	29.59*	34.38	**36.57**
28	35.43	36.02	33.52	38.6	32.64	**34.31**
29	30.67	31.38	31.46	29.64*	30.05	**30.49**
30	29.65*	30.47	30.44	29.90*	30.1	**30.20**
31	30.12	30.95	30.39	30.18	30.35	**30.85**
32	31.36	32.24	30.17	32.64	31.09	**31.35**
Cost [M.$]	6,073	6,195	7,000	6,056	6,220	**6,270**
N.of Eval.	-	-	16,910	53,000	25,402	**19,000**

Table 4.9.: Case study *D2b*: Comparison of the obtained solution with in literature reported solutions for the 3-loop network

and efficiency can be compared on the basis of identified solution values together with the needed number of function evaluations. In this respect, the developed method (last column in Table 4.9) manages to identify solution that is on the lower side of the needed function evaluations (*N.of Eval.*) and still has a very good minimum cost result ($Cost = 6,270M\$$). Additionally it is to be noticed that the minimum pressure of 30 m at all nodes present an additional limiting constraint that is not fully obeyed by all presented method (pressures under 30 m are marked with * in Table 4.9) but is satisfied by the here calculated solution.

Model Sensitivity - As just mentioned, the minimum pressure constraint has a very large influence on the final result of the optimisation procedure. In order to test and quantify this statement 4 new optimisation runs are made with the relaxed minimum pressure constraint. The results are shown in Figure 4.33. It can be seen that the optimisation procedure manage to identify the lower cost solutions for the weaker minimum pressure constraint scenarios. Although the relative improvements are quite modest (approximately 1 % of cost savings for 1 m lower pressure constraint), the possiblity to identify different solutions that are optimal for different minimum pressure scenarios could help in the creation of the design solutions that are better accommodated to the user's needs. For example some users and decisio makers may readily trade-off some savings in economic costs for lower distributed pressures. In addition with the developed model it is possible to define different minimum pressure constraint for different parts, or zones, of a water supply network.

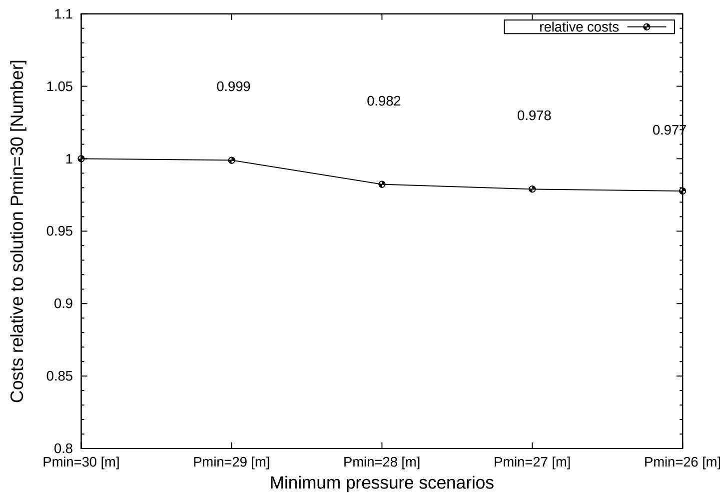


Figure 4.33.: Case study *D2b*: Relative total cost reduction for the relaxation of the minimum pressure constraint

Finally, as it was shown in Figur 4.30 on page 106, the performance failure analysis is very sensitive to the way how uncertainty of the parameters is defined and to the parameters' mutual dependency. Since the variations of uncertain parameters are very rarely independent it is very important to include their dependencies during creation of the samples for the testing of the performances and the reliability of a system. Furthermore looking at the results of the components failure analysis (Figure 4.22) it is obvious that selection of the component failure scenarios play a very important role. A very good understanding of the water supply system structure and function is necessary for the definition of meaningful failure scenarios.

Model Efficiency - The progress of the algorithm for both case studies is presented in Figure 4.34. Although it may be seen that the algorithm reach quite fast one near optimal solution, it needs much more computational effort until the whole set of solutions reaches the same optimum. This is due to the expansion of the Simulated Annealing algorithm to work with a set of solutions instead of with only one. Such costs in computational time can be accepted with the argumentation that the independent identification of the same optima from the whole set of solutions, is the way to increase the probability that the identified solution is a global optimum. Even more, this helps to distinguish among accidentally and systematically identified optima and improve the robustness of the algorithm. In order to further increase the chances to identify the global optimum and advance the exploration of the whole solution space, the set of initial (starting) solutions for the Simulated Annealing algorithm is randomly generated.

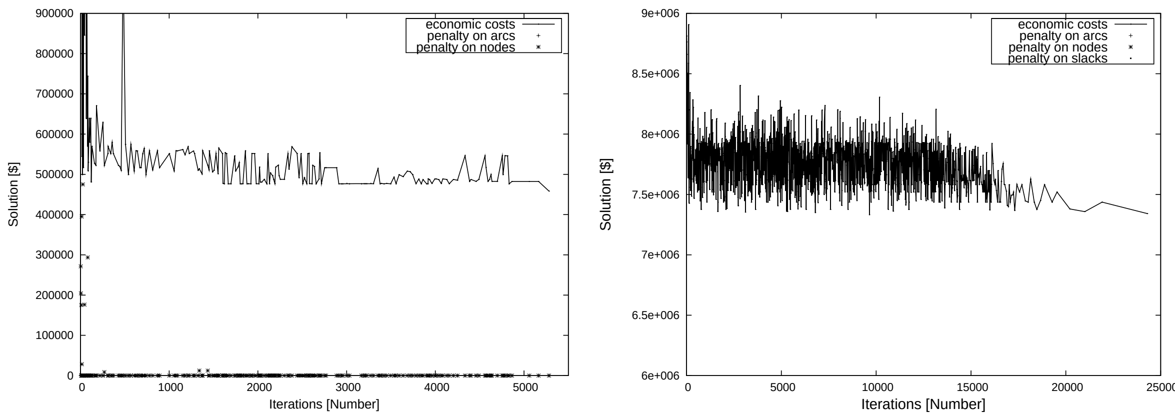


Figure 4.34.: Case study $D2a$ and $D2b$: Progress of the optimisation for the 2-loop and 3-loop network's optimisation

At the end it is important to mention that all presented methods, except the developed one, use the combination of the optimization model and the network simulator for the identification of the minimum cost solution. Such combination is used hare only during the identification of the system reliability and not during the optimisation itself. The exchange of data among these two models may significantly add upon the computational time. The developed method has an inherent network pressure calculator in a form of an algorithm that provides for the satisfaction of the downstream pressure conditions during the determination of the minimum cost diameters. Since, this internal network pressure calculation does not calculate the pressures in the whole network, for the system reliability assessment one network solver had to be coupled to the optimisation model, namely the solver of Gessler et al. (1985). The calculated solutions are tested in the EPANET (Rossman, 1993) network solver and proved to be satisfactory. The omission of the external network solver during the model design optimisation renders the developed model generally less computational time demanding and make it very applicable for large water supply networks.

4.3. Operation Model

Some of the aims of the analysis of the water supply systems' operation are to secure technical functioning of the systems, to provide the satisfaction of user's demands, to fulfil the regulatory criteria and engineering standards in terms of systems performances and services, to provide for system maintenance and further development, etc.. Obviously, for the achievement of all these aims, the operation analysis has to be done already during planning and design management stages. Only when incorporated in these early phases of water supply systems management the stated objectives of the system operation can be achieved later on (Walski et al., 2003).

4.3.1. Characterisation of the Operation Problem

In contrary to the design optimisation, which focuses on the worst stress conditions, the operation analysis primarily considers normal, or every day, operation conditions. Furthermore, the "steady-state analysis" for one specific point in time is not sufficient any more and the, so called, "extended period simulation" has to be done. This is actually the simulation of the system behaviour during some preselected time period. It enables the analysis of the components that transfer water in time, such as tanks and reservoirs, dimensioning of their capacities and definition of their operation rules. In addition, it provides for the creation of the operation rules for all manageable system components such as pumps, valves, pressure reducers, etc. Although each operation analysis is very case specific, from the engineering point of view following objectives can be stated:

1. Performance satisfaction.
2. Minimization of the investment costs for water storage elements.
3. Minimization of the operation costs for pump stations.

In addition to the performance indicators defined in the design phase (i.e. flows and pressures), the extended period simulation allows for the calculation of the water residual time and the volume exchange time in tanks and reservoirs. These indicators are the basis for the analysis of the water quality in water supply networks but since this study deal with the water quantity issues only, they will not be further considered. The performance indicators for the operation optimisation analysis are restricted to delivered flows, nodal pressures and volume exchange time in tanks.

As far as the second and third objectives are concerned, an additional restriction had to be introduced. Although for the majority of water supply systems the major part of the operation costs are the energy costs for water elevation and pumping, the exceptions are the systems that exclusively use gravity water flow. Operation costs mainly originate from the need to transport water along network and from the need to equalise temporal variation of water demands. Tanks and reservoirs are the elements that enable temporal redistribution of water by storing it in time. Since tanks also serve for the control and stabilisation of the pressures within a network, they are often set up above the other parts of the network and water has to be pumped to them. This cause pumping costs that often offer together with the investment costs in storage elements the main potential for the optimisation of system operation. If taken into account that the electricity is a primary energy source in almost all developed countries and most of the developing world in water supply systems and that the cost of electricity are almost always divided into different levels according to the time of consumption, one can identify a significant potential for the savings in operation costs by better accommodating the pumps operation schedules with the energy costs variations. The necessary prerequisite for the efficient use of water pumps is the existence of enough storage facilities that can accept, store and redistribute water in time. But, larger storage volumes increase the investment costs in tanks and reservoirs. The trade off among these two type of costs, keeping in mind the satisfaction of the performance criteria, is the prime focus of the applied water supply systems' operation analysis.

Accordingly, tanks capacities and pumps operation schedules are selected as the main decision variables of the operation optimization problem. The Minimum Cost Flow Network optimization problem, in its path form, is used again only the fixed and variable costs are accommodated to refer to the tanks investment costs and pumps operation costs, respectively. Furthermore, instead of the integer variable y_{ij} that referred to the existing and potential elements, two variables $y_{T_{ij}}$ and $y_{P_{ij}}$ are introduced to refer to tanks' capacity and pumps' operation schedule. Similar to the design problem defined in equation refẽq:mindsgn on page 92 this is an single-objective problem (minimisation of the economic costs). The addition of the time dimension significantly adds up on the model complexity since the flow vector is not any more a stationary value but instead the set of, in time ordered, flow vectors.

As previously stated, the constraints for such optimization problems are the user demands in terms of delivered flow and the engineering standards in terms of allowable pressures as well as the mass and energy conservation equations for the network flow. The general Minimum Cost Network Flow problem from the equation 3.36 on page 48 for the single-objective optimisation for the optimisation of system operation (minimisation of investment and operation costs) with two decision variables (tank capacities and pump schedules), can be rewritten as:

$$min. \quad z = \sum_{t \in T} \sum_{\pi_k \in \Pi} \sum_{a_{ij} \in \pi_k} (C_{P_{ij}}(x^{\pi_k^t}) y_{P_{ij}} + C_{T_{ij}}(x^{\pi_k^t}) y_{T_{ij}}) \tag{4.6}$$

subject to:

$$\begin{array}{lll} \sum\limits_{n_j : a_{ij} \in A} x_{ij}^t - \sum\limits_{n_j : a_{ji} \in A} x_{ji}^t = b_j^t & \forall n_j \in N & \forall t \in T \\ \sum\limits_{\pi} \delta_{ij}^{\pi} x^{\pi^t} \leq \kappa_{ij} y_{ij} & \forall \pi \in \Pi & \forall t \in T \\ p_{min_j} \leq p_j \leq p_{max_j} & \forall n_j \in N & \\ y_{P_{ij}} = (0 \vee 1)^t, \; y_{min_{T_{ij}}} \leq y_{T_{ij}} \leq y_{max_{T_{ij}}} & \forall a_{ij} \in A & \end{array} \tag{4.7}$$

where x^{π^t} is a path flow on a conforming simple path π in the time period t for which the individual arc flows can be obtained as $x_{ij}^t = \sum\limits_{\pi} \delta_{ij}^{\pi} x^{\pi^t}$ for $\delta_{ij}^{\pi} = 1$ if a_{ij} is on path x^{π} and 0 otherwise. C_P and C_T are pump operation and tank investment costs functions and correspond to the pump schedule expressed as a timely ordered set of $y_{P_{ij}} = (0 \vee 1)^t$ (0 if pump $y_{P_{ij}}$ is turned off and 1 if it is turned on) and tanks capacities $y_{T_{ij}}$. Parameter κ_{ij} stands for the upper capacity limit of the arc a_{ij} while p_{min_j} and p_{max_j} stand for minimum and maximum pressures at the node n_j, respectively. Finally the collection of $\mathbf{x^t} = \{x^{\pi_k^t} \mid \pi_k \in \Pi\}$, from all conforming paths Π, represents timely ordered set of network flow vectors for an ordered set T of all time periods.

Since the pump operation and the tank investment costs are both evaluated in the same units (i.e. money) this is a strictly speaking single-objective optimization problem. Nevertheless it consists of the two decision variables sets, first are the tank capacities and second are the pumping schedules. Both variables are discrete and the pumping schedules have an additional dimension since they are distributed in time. Both variables are connected to the network flows but this time the network flows have the time dimension. Although, adding of the time component significantly adds on the optimisation problem complexity the same methodological concept as for the planning and design problem, is accommodated and applied for the operation problem, too.

4.3.2. Accommodation of the Solution Methodology

A solution technique for the defined operation optimization problem should be capable of efficiently dealing with the following main tasks:

1. Representation of the water supply system operation.
2. Examination of the various tank configurations and sizes.
3. Identification of the minimum cost pumping schedules.

In addition to the representation of the system structure, the ability to represent the functioning of pumps, tanks, valves and other flow and pressure control facilities is of the main importance for the proper operation analysis. This is not any more just the calculation of the flows and pressures in the network but often the implementation of very complex operating rules for the opening and closing of valves, turning on and off of the pumps, activation of the booster stations for pressure increase or reducing valves for pressure decrease, etc.

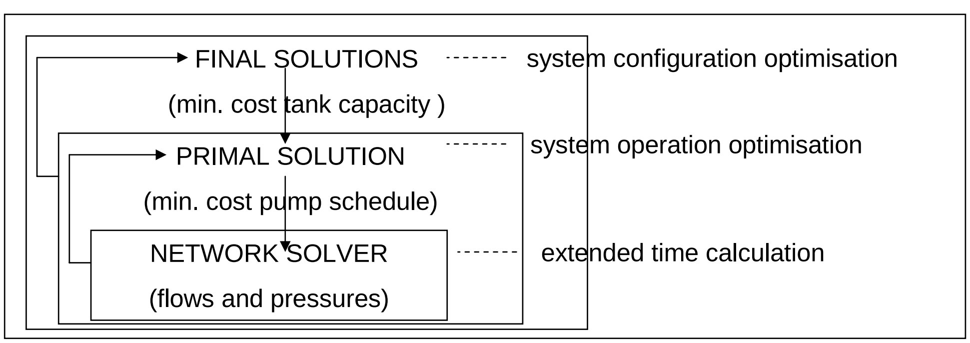


Figure 4.35.: Integration of Network Solver in the operation optimisation model

Since this study focus on the optimization of the pumping schedules and tank configuration and size, a network solver of Gessler et al. (1985) is adopted and coupled with the optimisation model for the calculation of network flows, pressures and tank levels during the extended period simulation as in Figure 4.35.

The optimization solution procedure consists of the following steps (Figure 4.36):

1. *Input* - Beside basic data about a water supply system (i.e. configuration, layout, capacities, supplies, demands, hydraulic properties, etc.) the data for the temporal water distribution have to be defined, too. These are mainly the position, available volumes, and operation water levels of tanks and reservoirs as well as the position and characteristics of pumps and pressure reducing valves. In addition, investment costs of the new elements or elements that can be rehabilitated as well as the energy cost of pump operation have to be provided. Finally the time period for which the extended period simulation is to be done, have to be defined. It depends on the system characteristics and size but it is mainly defined as the period for which the tanks cover the consumption variation (normally 24 hours or daily demand variation).

2. *Primal solution - system operation optimization* - The Simulated Annealing algorithm is used to identify the minimum cost pump operation schedule for one system configuration with predefined tank and reservoir volumes. The algorithm iteratively produces random pumping schedules, calculate energy usage and corresponding costs and accepts or rejects solutions based on the metropolis schedule. The identification of the network flows and pressures at each time step of the extended period simulation is done by the network solver of Gessler et al. (1985). Identified minimum cost solution is referred as the primal solution and is used as the reference point for the identification of possible operation savings by investing in tank and reservoir expansion or building of some new storage elements.

3. *Final solutions - system configuration optimization* - The Branch and Bound algorithm is used to question different combinations of tank and reservoir volumes and the above described Simulated Annealing procedure to identify the minimum cost operation schedule for each combination. The total costs (sum of investment and operation costs) are calculated and compared until the minimum cost configuration is found. In order to avoid analysis of all possible combinations of new elements, the algorithm sorts different combinations of tanks and reservoirs with their possible volumes in a tree ordered structure that enable omission, so called "fathoming", of configurations that are only refinements of the already examined ones. Since the investment and operation costs are referred to the primal solution (costs are divided with the corresponding costs of the primal solution) the progress of the algorithm can be easily followed. The finally identified solution is a trade off among tank investments and pump operation costs that yield a minim total costs for a given operation period.

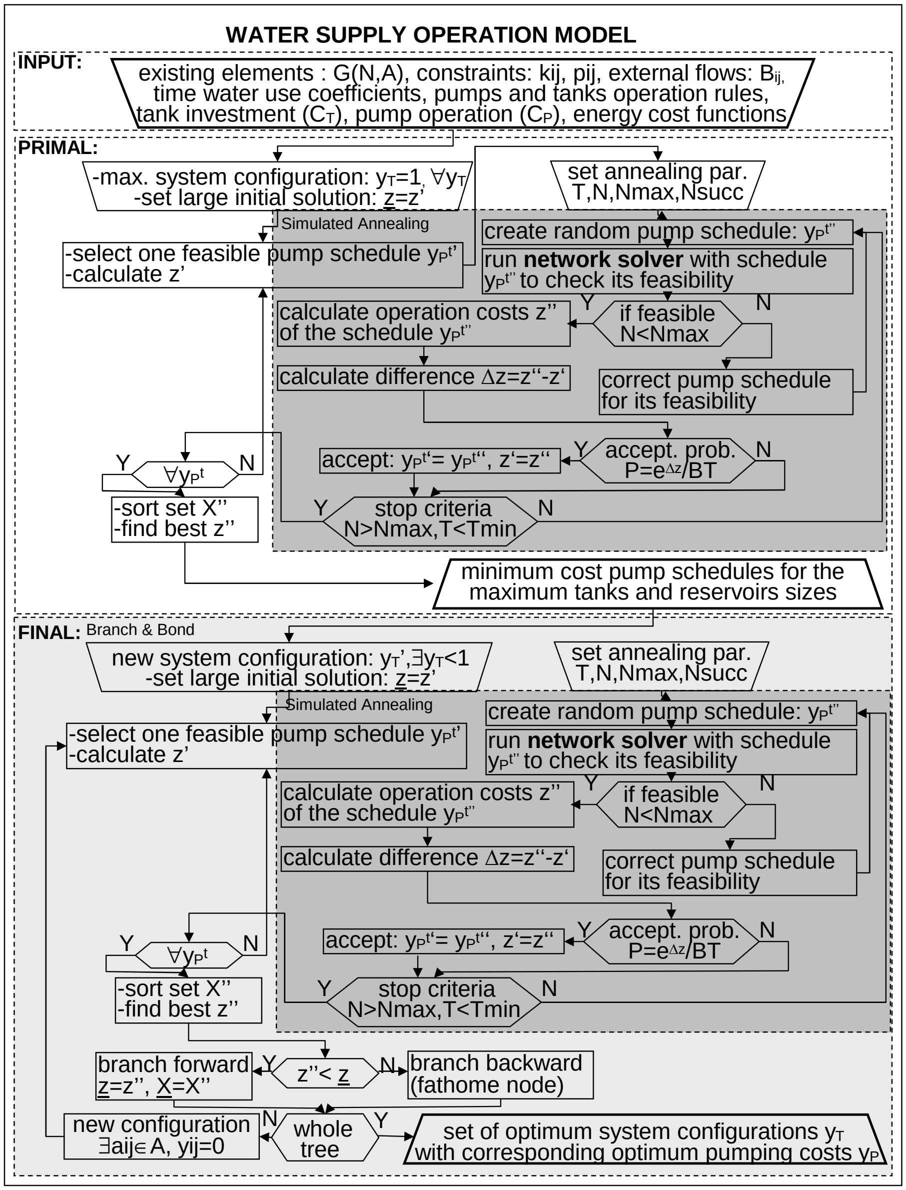


Figure 4.36.: Flow chart of the operation model

4.3.3. Case Study $O1$ - Operation Model Demonstration

Study Description - In order to present the purpose of the operation model the case study of Alperovits and Shamir (1977) is used once more. Since the original problem does not include the operation costs they are added based on the case study of Walski et al. (1987). In order to make the network of Alperovits and Shamir (1977) more interesting and convenient for the operation analysis one pump and one water tank are added. Since the main aim is only the demonstration of the model function, the characteristics of the network and costs functions are intentionally left as simple as possible, while the next case study is used for the testing of the model's capabilities on an well know optimization problem of Walski et al.

(1987). The configuration of the network with the added pump node ($N11$) and tank node ($N12$) is presented in Figure 4.37. The identified minimum cost diameters in the design stage optimisation are adopted as the existing network configuration. Beside diameters the rated[16] flow and pressure for the pump *A11* are adopted to be 1000 m^3/day and 150 m.

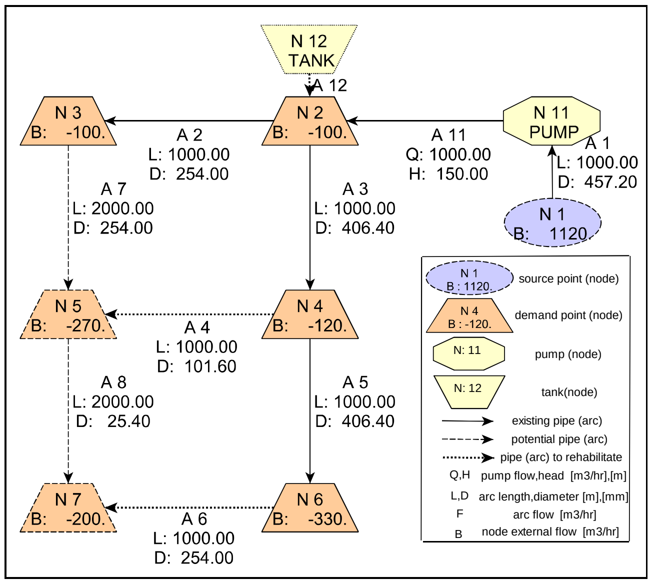


Figure 4.37.: Case study $O1$: Network Configuration [adapted Alperovits and Shamir (1977)]

Problem Statement - In addition to the network description in subchapter 4.1.3 (page 72) and the cost functions defined in subchapter 4.2.3 (page 97) the daily water demand variations and the daily variations in energy costs had to be adopted. Since the 24 hours operation is selected to be the governing time period for the system operation, the daily demand variation is adopted as in Walski et al. (1987) and presented at the left graph in Figure 4.38. At the right graph in Figure 4.38, a typical daily partitioning of the industry electricity costs in a 3 phase system (normal, on-peak and off-peak) is given.

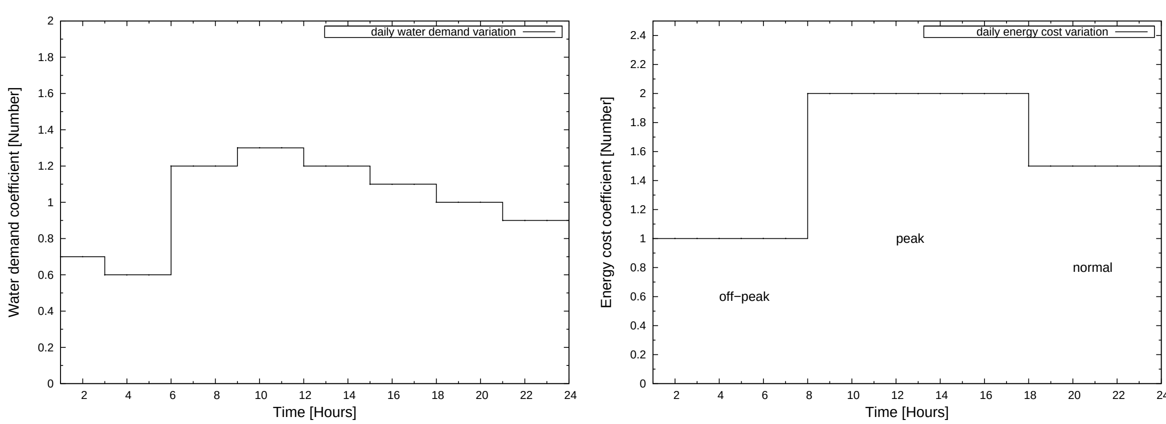


Figure 4.38.: Case study $O1$: Adopted water demand coefficient [as in Walski et al. (1987)] and energy cost coefficient [typical 3-phase partitioning]

[16]flow and head at which maximum pump efficiency is achieved

The average electricity cost for the analysis time period are adopted as in Walski et al. (1987) and are projected to their net present value using following cash flow calculation:

- electricity cost: $L_o = L_n(1 + i/100)^n$

where L_o is the net present value of the electricity cost, $L_n = 0.12\ \$/kWh$ is the average electricity cost in time period t, $i = 12\ \%$ is the interest rate for the time period and $n = 20\ years$ is the number of years for which the cash flow discounting is done.

The pump characteristic and the pump efficiency are also adopted from Walski et al. (1987) and are simplified to the following polynomial dependencies:

- pump characteristics: $H = -2 * 10^{-6} * Q^2 - 7 + 10^{-4} * Q + 300.31$
- pump efficiency: $E = -3 * 10^{-6} * Q^2 + 0.0264 * Q + 2.8571$
- pump power input: $P = 9.81 * Q * H * E$

where Q and H are rated flow and head of the pump, E its wire-to-water efficiency[17] and P is the the electrical input to the motor of the pump[18].

The pump operation costs are calculated as:

- pump operation costs: $C_P^{opr} = L_o * P$

where in order to calculate the power input P, the work of the pump have to be divided into the time intervals with constant head and flow values. These are initially set up as 1 *hour* intervals but are automatically shorten in cases of the earlier change of the pump working mode that are driven by tank water levels.

Finally, the investments costs of pump stations and the investment costs of tanks are also defined as in Walski et al. (1987):

- pump investment costs (new pumps): $C_P^{inv} = 500 * Q^{0.7} * H^{0.4}$
- pump investment costs (rehabilitation of existing): $C_P^{reh} = 350 * Q^{0.7} * H^{0.4}$
- tank investment costs (new tanks): $C_T^{inv} = -5 * 10^{-7} * V^2 + 0.9853 * V + 68800$
- tank investment costs (rehabilitation of existing): $C_T^{reh} = 0.3 * C_T^{inv}$

where Q and H are rated flow and head of a pump and V and A are volume and area of a tank. Since water levels in the storage units often regulate network pressures and have fixed operational levels (e.g. minimum level, level for the start of the pump, maximum level, etc.), for the calculation of the tank investment costs the area of a tank is much more suitable then its volume. The equation then transforms to $C_T^{inv} = -0.016 * A^2 + 184.26 * A + 68800$

[17] the ratio of the energy delivered by the pump to the energy supplied to the input side of the motor
[18] power input as a measure of the rate at which work is done

and enable easier optimization of the tank size without changing of the pressure conditions. For the presented case study the minimum and maximum water level in the tank $N12$ are adopted as 15 m and 60 m, and the minimum and maximum tank area as 50 m^2 and 100 m^2. In order to be consistent with the defined operation optimisation problem (Equation 4.6) total pump and tank costs are obtained as:

- pump costs: $C_P = C_P^{inv} + C_P^{opr}$
- tank costs: $C_T = C_T^{inv}$

Primal solution - Pumping Schedule Optimisation - From the operation point of view the most cost demanding elements of water supply systems are pump stations. Therefore the first step of the optimization procedure focuses on the identification of the minimum cost pumping schedule for the given daily distribution of water demands and the predefined available reservoir's and tank's capacities and minimum and maximum pressure conditions. Knowing that the pumping costs are for majority of water supply systems actually the cost of the electricity used during the operation of pumps, the problem can be reduced to the identification of the pumping schedules that can cover for given water demands variations by filling of existing storage capacities mainly during the time periods of lower energy cost. Furthermore, since the needed energy input and the efficiency of the pump operation depend on the flow and head characteristics during its work, in many water supply systems the pumps are either used in their optimal working regime or turned off. Namely installation of larger number of smaller pumps enables the regulation of the pumping regime by turning some pumps on or off and allowing them to work only in high efficiency range. Such simplification is not appropriate for the pumps that can modify their optimal working range (e.g. variable speed pumps, variable blade pumps, etc.) and for the pumps that serve for the pressure increase (buster stations). Nevertheless, the attention of this study is on the pumps that serve for the balancing of the water demand variations since they are the ones where the most cost optimisation potential exists.

For a system with N pumps and M time intervals where each pump can take either an "on" or an "off" state in each time interval, the total number of operation modes combinations is 2^{N^M}. Since the feasibility of an individual pumping schedule depends on the water levels in its controlling water tank (for pressure controlled pumps reaching of a certain boundary head on some predefined network nodes causes either start or stop of the pump operation) and the feasibility of the whole schedule depends on the satisfaction of the water demands and nodal pressures in the whole network, many of the pumps operation combinations will yield infeasible solutions. Nevertheless, since it is very hard to a priori eliminate the infeasible combinations, the problem is NP-hardto solve. Still the applied Simulated Annealing algorithm with a random selection of the pumps operation modes for some predefined time intervals is able to deal with such a problem. For the above given water supply network (Figure 4.37), the optimal pump operation schedule and tank water level in each hour of the 24 $hours$ simulation with an adopted tank area of 50 m^2 and minimum and maximum tank water levels of 15 and 60 m are presented in Figure 4.39. In addition the corresponding energy cost and water demand coefficients are given in the same Figure.

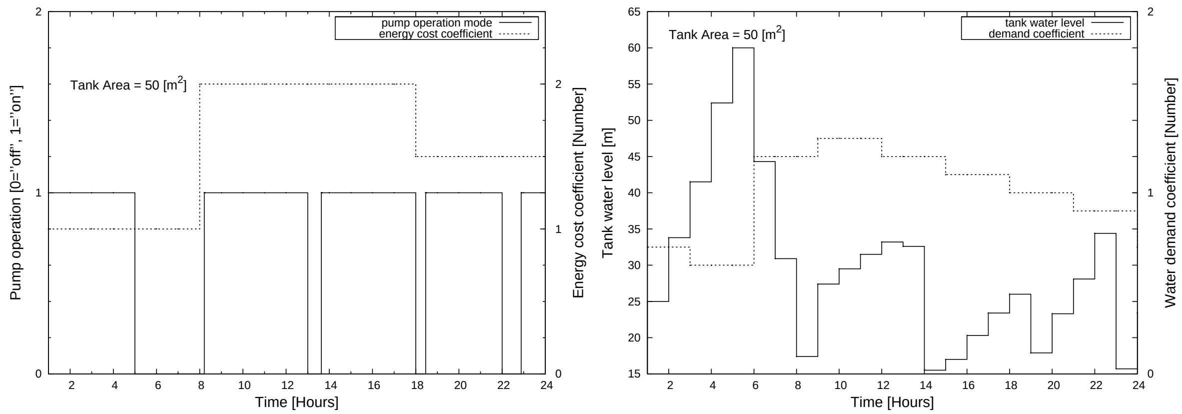


Figure 4.39.: Case study $O1$: Obtained pump operation schedule and tank water level for the primal solution [tank area of 50 m^2]

The simulation is started at 0 $hours$ with water level in the tank $N12$ at 25 m. The identified optimal pumping schedule fills tank $N12$ in the first 5 $hours$ until its maximum capacity (water level = 60 m) is reached. These are the off-peak energy hours and although they last until 8 $hours$ they can not be used more while the capacity limitation of the tank $N12$ to the 50 m^2 $*$ 45 m = 2250 m^3 is already reached and the water demand in this period is too low to empty the tank. Only after the water demand increases in next 3 $hours$ the tank $N12$ is partially exhausted and the pump $N11$ can be turned on. Although these are the the peak energy cost hours, the identified operation schedule fill the tank $N12$ only to the minimum amount necessary to satisfy high water demands in this period. In the following period of normal energy costs (after 18 $hours$) the pump $N11$ operates in "on" mode filling the tank $N12$ for the next day consumption. It is to be noticed that the algorithm is started with the water level in the tank at 25m and ends with the tank water level of 15m that is the minimum allowable value. Although this indicates the optimal use of the tank volume this would not be allowable for many real life system and these two values should be additionally optimised for specific applications.

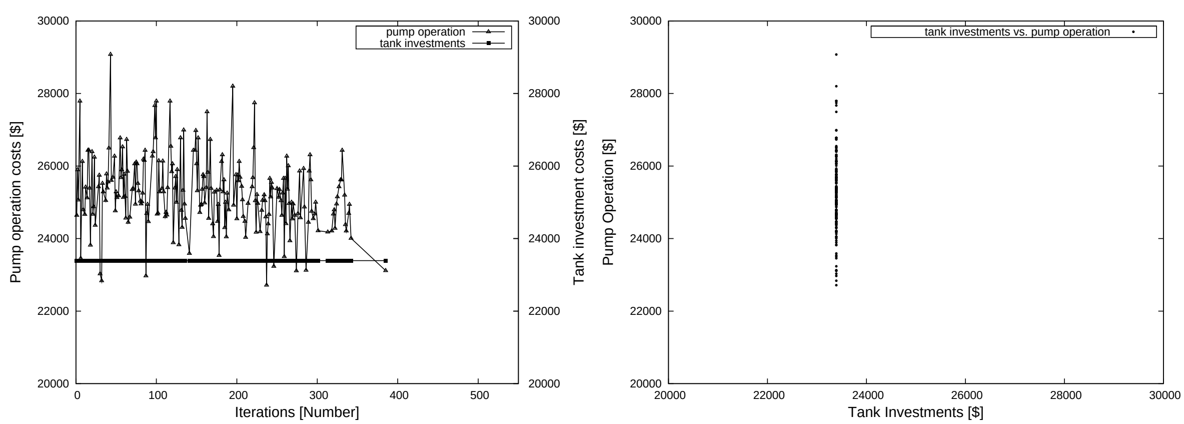


Figure 4.40.: Case study $O1$: Identified tank investment and pump operation costs values during single-objective optimisation

The single-objectivity of the identified primal solution can be noticed in Figure 4.40. The investment costs are hold constant and the improvements are obtained only on the operation costs. Obviously this does not allow for the trade-off among these two and have to be encompassed in the final solution.

Final Solution - Tank Area Optimisation - As presented, the optimization of the pumping schedule in the primal solution was primarily constrained by the available storage capacity of the tank. Therefore it is necessary to jointly optimise the investments in tank storage volumes and the pump operation costs. Although both these values are expressed in the same units (i.e. money) they refer to different times and have to be brought to the same point in time. In addition, in order to enable easier comparison of the already identified minimum cost pumping schedule for the adopted minimum tank area of 50 m^2 with the new solutions, all new solutions are referenced to it by dividing their investment and operation costs with the investment and operation costs of the primal solution.

Due to the fact that the building requirements for the water tanks and reservoirs often demand for step-wise defined capacities, the tank capacities are adopted to be a discrete decision variable. For the purpose of presenting of the developed model, 10 % increases from the existing, or minimum capacity of some tank are considered. If the maximum capacity increase of 100 % is adopted, 10 possible tank sizes have to be questioned for each tank. For a system with K tanks the addition to the problem complexity is then 10^K. The identified final optimal solution in terms of tank sizes and pumping schedules is presented in Figure 4.41.

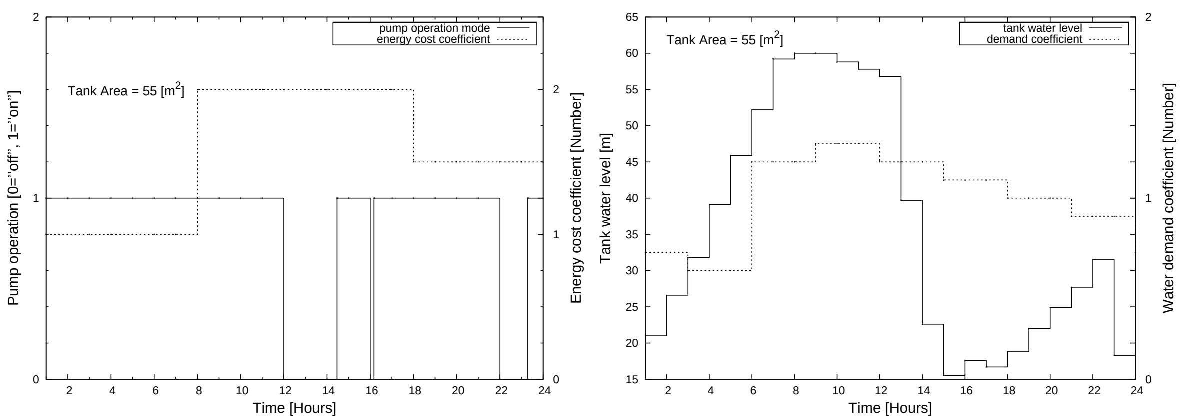


Figure 4.41.: Case study $O1$: Obtained pump operation schedule and tank water level for the final solution [tank area of 55 m^2]

When compared to the primal solution, shown in Figure 4.39, the tank level and the pumping schedule of the final solution show very promising results. Already an increase of 10 % in the tank $N12$ area (from 50 to 55 m^2) provided for the much better tank filling during the off-peak energy hours. Similar as for the primal solution, the minimum cost pumping schedule starts with the pump operation for the first energy off-peak period until the full capacity of the tank $N12$ is reached. Since the pump now need 7 $hours$ to completely fill the tank $N12$, the high water demands in the first period of the peak energy cost cause that the pump stays in "on" mode for the next 4 $hours$. The total capacity of the tank ($2475m^3$) is still relatively small (approximate 10%) in comparison to the total water demand ($26880m^3$). The pump is then turned off until the whole volume of the tank is exhausted (minimum level of 15 m is reached). Since this happens in less than 3 *hours* the pump is again turned on. The pumping during the normal energy cost hours (from 18 to 24 *hours*) is scheduled in a way to keep the water level in the tank $N12$ in the lower range ending up with the almost empty tank at the end of the simulation. This can be easily corrected either by defining one end tank water level constraint or by prolonging the duration of the extended time simulation.

A further expansion of the storage area would most probably allow for even a better pumping schedule but would also cause higher tank investment cost values. In order to illustrate the opportunism among tank investments and pump operation costs all accepted solutions during optimisation are presented in Figure 4.42. The left graph presents the values of tank investments and pump operation costs for all accepted solutions along the progress of the algorithm while the right graph presents the mutual relation among these two costs.

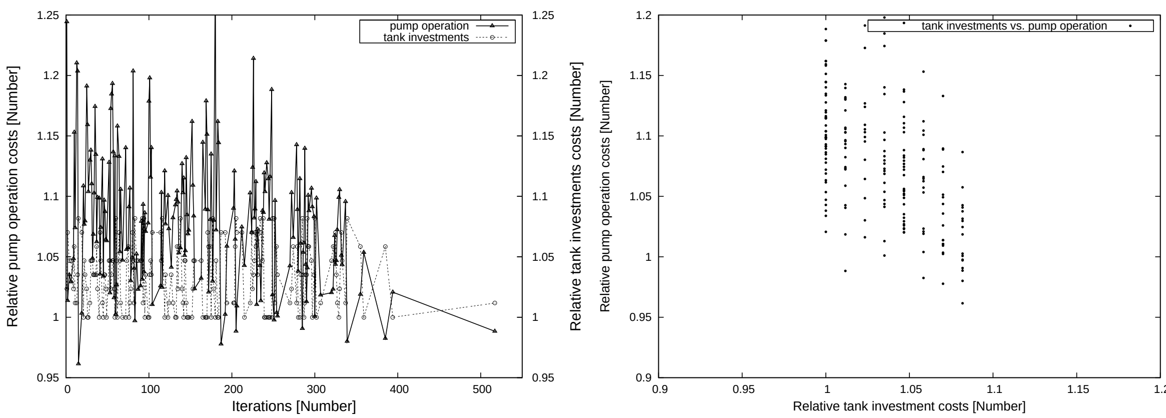


Figure 4.42.: Case study $O1$: Identified tank investment and pump operation costs values during multi-objective optimisation

As obvious from the left graph in Figure 4.42 the smaller tank investment costs cause higher pump operation costs and low pump operation costs can be identified only for the high tank investment costs. If compared directly, these two types of costs form a Pareto-set whose Pareto-front of optimal solutions has a form of an almost straight line. The best identified solution is the one that slightly outcomes the others on this line. Most probably the improving of the Simulated Annealing parameters (e.g. "cooling schedule", "neighbourhood function", etc.) would enable even the identification of some better solutions. In order to stress once more the lack of theoretical proofs of the global optimality of the method and to warn once more from the care-less and too trust-worthy use of the algorithm such improvements are deliberately omitted in this study.

In order to reduce the questioning of the not-optimal combinations, the Branch and Bound technique is used. This tree based optimisation technique helps to avoid the unnecessary examination of the combinations that yield solutions that can not be better than the already found ones. The Branch and Bound tree is created in a way that the maximum tank capacities are set up at the upper branches and are gradually reduced by developing a hierarchical structure of the tree. If the optimal identified pump operation schedule has worse costs than, at that point, the best found schedule, than the whole branch with the smaller tank capacities can be avoided since it can yield only worse solutions in terms of the pump operation costs.

The optimisation of the pumping schedules for each system configuration is achieved with the Simulated Annealing algorithm.

At the end, the minimum calculated pressures within the network for the primal and the final solution during the whole simulation period are shown in Figure 4.43. Since the maximum pressures for this case study, are controlled by the tank water level they are always below the maximum limit and only the minimum pressures are shown. This is interesting since the pressures are not modelled as a rigorous constraint, but instead the solution is penalized if the calculated pressures avoid their limitations. The pressure distribution can be, in a way, considered as an indicator of the validity of the solution. It is to be noticed that for both solutions the minimum pressure values stay above the predefined limit of 35 m.

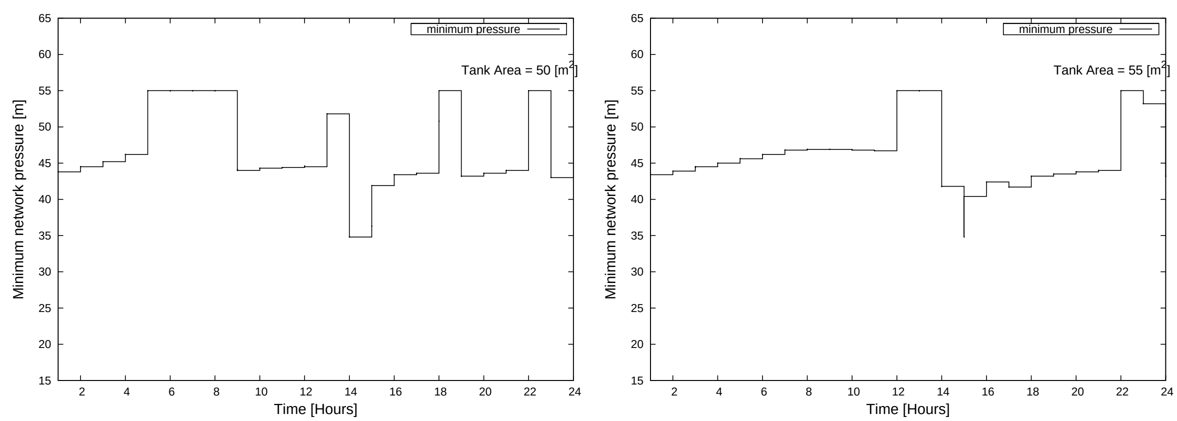


Figure 4.43.: Case study $O1$: Obtained minimum pressures for the primal and the final solution

4.3.4. Case Study $O2$ - Operation Model Validation

For the purpose of the testing of efficiency of the developed operation model, it has been applied on the, so called, Anytown network developed by Walski et al. (1987). This hypothetical water supply system is built for the purpose of testing and benchmarking different water distribution network design optimization models and is the key reference case study in the water supply research literature. Since the problem of the selection of the optimal pipe diameters has been already addressed by many other researches (Walski et al., 1987), the problem of selection of the optimal tank position and sizes, has been a main focus of this study. Due to the good data availability, the case study was easily accommodated for the application of the operation model.

Study Description - Anytown represents a typical small town water supply system that takes water at a river intake, treats it at a central plant and pumps it, with three parallel pumps, to the distribution network as in Figure 4.44. The distribution system itself consists of the old part in the central city, with cast iron pipes, and two new housing and industrial areas to the north-east and west, respectively with plastic pipes. Two existing elevated tanks ($N65$ and $N165$) each with capacity of 250.000 *gallon* (approx. 1136 m^3) are aimed to provide for the daily water and pressure inequalities and are a bit small for the system of this size

(Walski et al., 1987). Due to the increased industrial consumption in the western part of the town the water supply utility has a problem to fill the tank erected there ($N165$) and considers either to upgrade the existing tanks or to build a new one at one of the locations: $N85$, $N145$ or $N155$. The selection of the position of a new tank as well as the determination of its capacity, in a way to provide the optimum among tank investments and future pumping costs, is the main problem to be dealt with in this study.

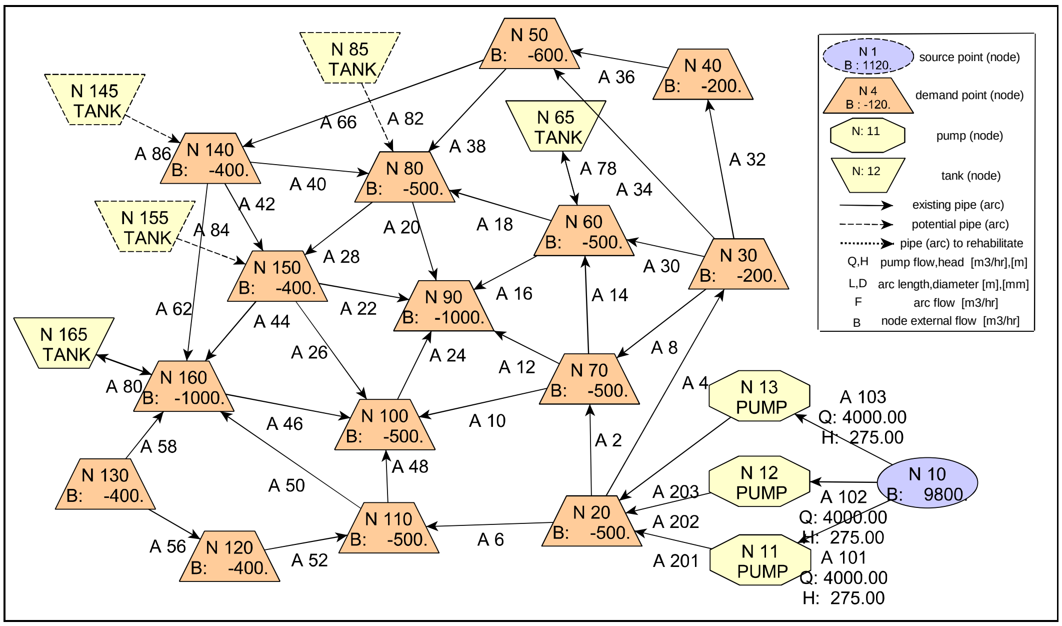


Figure 4.44.: Case study $O2$: Network configuration [adaptation from Walski et al. (1987)]

The detailed characteristics of the network that are of importance for the application of the operation model are given in tables 4.10 and 4.11[19]. The diameter (D), length (L) and the friction coefficient value (C) are given for all transport network pipes as well as for the pipes that connect elevated tanks with the rest of the network, so called risers: *A78, A80, A82, A84, A86*. Network nodes are defined with their elevation (Z) and projected average water consumption (Q). Both existing (*N65* and *N165*) and potential *N85, N145, N155* elevated tanks are given with their position and current area (A). Finally for all three pumps, rated[20] flow (Q) and head (H) are given. All arcs and nodes are referenced with their original identification number as in Walski et al. (1987).

[19] pipe diameters, lengths and flow in arcs, elevation and external flow in nodes, area of the tanks and rated height and flow in pumps are given in American measurement units as in original problem of Walski et al. (1987) but can be easily converted to the SI-units using: 1 $in = 0.0254\ m$, 1 $ft = 0.3048\ m$, 1 $gpm = 0.000067\ m^3/s$

[20] flow and head at which maximum pump efficiency is achieved

Arc ID	2	4	6	8	10	12	14	16	18	20	22	24	26	28	30	32
D [in]	16	12	12	12	12	10	12	10	12	10	10	10	12	10	10	10
L [ft]	12000	12000	12000	9000	6000	6000	6000	6000	6000	6000	6000	6000	6000	6000	6000	6000
C	70	120	70	70	70	70	70	70	70	70	70	70	70	70	120	120
Arc ID	34	36	38	40	42	44	46	48	50	52	56	58	60	62	64	66
D [in]	10	10	10	10	8	8	8	8	10	8	8	10	8	8	8	8
L [ft]	9000	6000	6000	6000	6000	6000	6000	6000	6000	6000	6000	6000	6000	6000	12000	12000
C	120	120	120	120	120	120	120	70	120	120	120	120	120	120	120	120

Riser	78	80	82	84	86
D [in]	12	12	12	12	12
L [ft]	100	100	100	100	100
C	120	120	120	120	120

Pump	org.	101	102	103	201	202	203
D [in]		pump	pump	pump	pump	pump	pump
L [ft]		in	in	in	out	out	out
C							

Table 4.10.: Case study $O2$: Characteristics of network arcs [adaptation of Walski et al. (1987)]

Nod	10	20	30	40	50	60	70	80	90	100	110	120	130	140	150	160	170
Z [ft]	10	20	50	50	50	50	50	50	50	50	50	120	120	80	120	120	120
Q		500	200	200	200	500	500	500	1000	500	500	200	200	200	200	800	200

Tank	65	165	85	145	155
Z [ft]	215	215	215	215	215
Area	1000	1000	-	-	-

Pump	org.	11	12	13
Q		4000	4000	4000
H [ft]		275	275	275

Table 4.11.: Case study $O2$: Characteristics of network nodes [adaptation from Walski et al. (1987)]

In addition to the balancing of the daily water demands variations, the tanks serve to provide for the stable distribution of pressures and should be operated between elevation of 225 ft (approx. 68.6 m) and 250 ft (approx. 76.2 m). Since the tanks are placed at the elevation of 215 ft the water level in tanks have to be be kept between 15 ft and 35 ft. In order to accommodate for these fixed lower and upper water levels in tanks, instead of tank volumes, the area of a tank is defined as a decision variable. A further simplification is achieved by adopting that the pump curves of Walski et al. (1987) defined with 4 points can be substituted with quadratic polynomial function in which the maximum head is defined as 33 % greater then the rated one and the maximum flow as the double of the rated one ($H_{max} = 1.33H_r, Q \to 0$ and $Q_{max} = 2Q_r, H \to 0$) .

The water demand pattern is the same as given in the previous case study in Figure 4.38 on page 118. and is allocated to all demand nodes except to the node *N160* that is a large industrial consumer. This consumer demands a constant amount of water during the first and second working shift form 8 to 22 $hours$ (demand coefficient equals 1) and reduces its consumption for 20 % during the night hours (demand coefficient equals 0.8). Furthermore, the daily energy cost coefficients given in Figure 4.38 on page 118 are adopted for this case study, too. The net present value of the electricity cost is calculated as in the original problem of Walski et al. (1987). The equations are also already provided on page 119. The average electricity cost, the infation rate and the time period also stay the same: $L_n = 0.12$ \$/kWh, $i = 12$ % and $n = 20$ $years$. Finally the pumps wire-to-water efficiency, the investment costs for the construction of new or the rehabilitation of existing tanks and pumps and the pumps operation costs are also taken from the original problem formulation and are presented in the previous case study on page 119. All other parameters given in the Walski et al. (1987), that are not relevant for the defined operation problem, are omitted.

Problem Statement - When analysing the results of the different optimization models in his "Battle of the Network Models: Epiloque", Walski et al. (1987) concluded that most of the differences among offered optimized solutions could be traced to the position of the storage tanks. Although all offered solutions featured the addition of at least one new tank the selection of its size and position was made a priori to the optimization process and was exclusively based on the engineering judgement of the authors. Therefore a special attention will be devoted exactly to the problem of selection and dimensioning of water tanks with an optimisation model. Even more for many existing water supply systems the problem of trading-off among the investments in new water storages and the savings in pumping energy costs is of the prime importance from the operation point of view.

As it can be seen in Figure 4.44, in order to keep the solution of the problem traceable, only three potential locations for new tanks are considered here: *N85, N145, N155.* The possible area for each of them are defined from 500 ft^2 (half of the size of the existing tanks) to the 2000 ft^2 (double size of the existing tanks). These sizes are divided into 10 % increases, which make 15 possible sizes for each tank. These 15 possible sizes for three new tanks make 15^3 possible combinations in total. In order to reduce the problem size the Branch and Bound algorithm is used to structure the problem in a way to examine first the combinations with and withouth individual tanks, that is basically a reduced problem with the complexity of 2^3, and only then to determine the tank sizes for the selected tanks in combination with the selection of the optimal pumping schedules with the Simulated Annealing algorithm. In addition, it is not to be forgoten that the scheduling of N pumps in M time intervals in "on" or "off" mode yields a 2^{NM} combinatorial problem.

Primal solution - Pumping Schedule Optimisation - The first step of the optimization method uses the Simulated Annealing routine to identify the minimum cost pumping schedule for the fixed system configuration with all potential tanks expanded to their maximum capacities. The identified solution, referred as the primal solution, is dependent only on the pump operation costs. The identified minimum cost pumps operation schedule and tank water levels are presented in Figure 4.45.

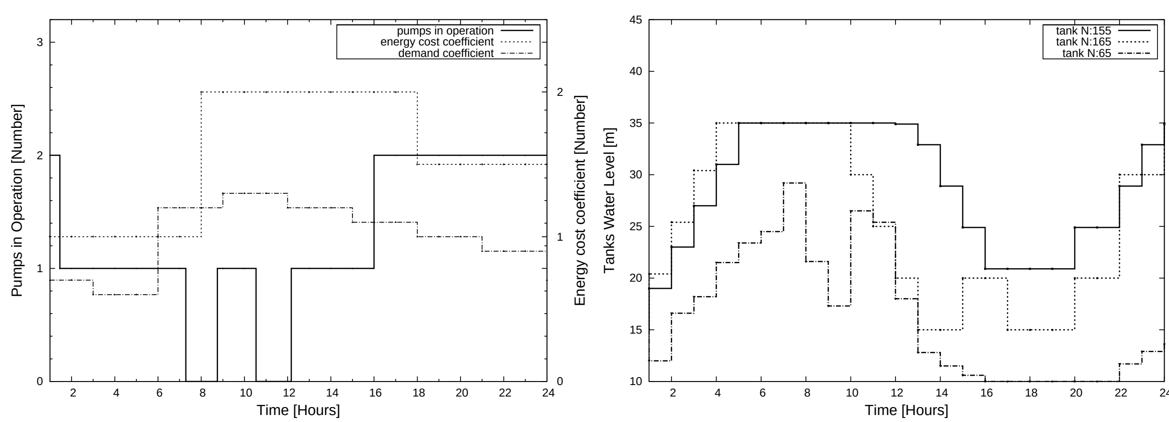


Figure 4.45.: Case study $O2$: Obtained pump operation schedule and tank water levels for the primal solution

As it can be seen from the left graph, quite large storage capacities provide for the use of only one or the most two pumps at the same time during the whole 24 *hours* operation. The allocated large capacity of the water tank $N155$ (2000 ft^2) provides for the enough water volume, that can be stored during the off-peak (0 - 8 *hours*) and normal (18 - 24 *hours*) energy hours, to cover the daily variations of demand in the western part of the network (the right graph in Figure 4.45). Nevertheless due to the quite small existing capacity of the tank $N65$, water have to be pumped to the central part of the network, even during the peak energy hours (8 - 18 *hours*). Looking at the water levels in potential tanks, it can be noticed that they all have one period of filling and one period of emptying. Furthermore, their large capacity provide for the quite large period of operation with full capacity.

Final Solution - Tank Area Optimisation - For the creation of the final combination of the potential tanks positions and sizes with the optimal pumping schedules, the combination of the Branch and Bound algorithm and the Simulated Annealing has been employed. Since the pumping costs are inversely proportional to the tank investment costs, when for one combination of the tank positions and sizes the pumping operation cost become higher than already identified minimum then all further refinements of this network configuration can be omitted from further questioning. The identified final solution suggests the addition of the tank *N155* with the area size of 655 ft^2 as an optimum among investment and operation costs. Together with pipe diameters and flows in the last time unit, this solution is presented in Figure 4.46.

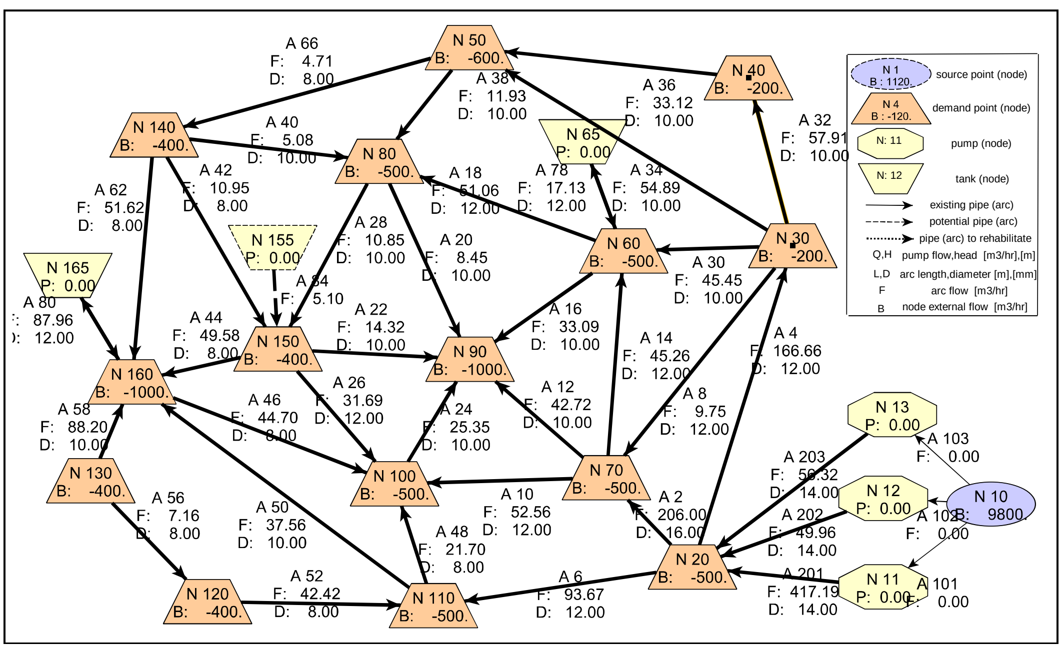


Figure 4.46.: Case study $O2$: Identified final solution

It is assumed that the storage tank *N155* has been selected due to its convenient position in between the nodes with the greatest consumption (*N90* and *N160*). Such a position provides for minimum head losses in supplying these two nodes and for the good pressure distribution around them. Furthermore its area of $655 ft^2$ (approx. $60 m^2$) and corresponding volume of $22925 ft^3$ (approx. $650 m^3$) make the addition of the 32 % to the total existing storage capacities. Although one could assume that the new storage capacities should be so large to enable fully satisfaction of the peak daily consumption with the pumping schedule that use only off-peak energy hours, due to the fact that the tank investment costs have the prevailing influence on the total costs, this is not the case. The identified optimal pumping schedule and the corresponding water level in tanks *N65,N165* and *N155* are shown in Figure 4.47. The selected minimum cost pumping schedule primarily uses the evening and night hours. Nevertheless, the storage capacities are still not large enough to cover the whole peak demand withouth pumping during the peak-hours. It can be seen that the water volume of all three tanks are quite equally used and their oscilations are quite modest. This brings an additional stability in the presseure distribution within the network.

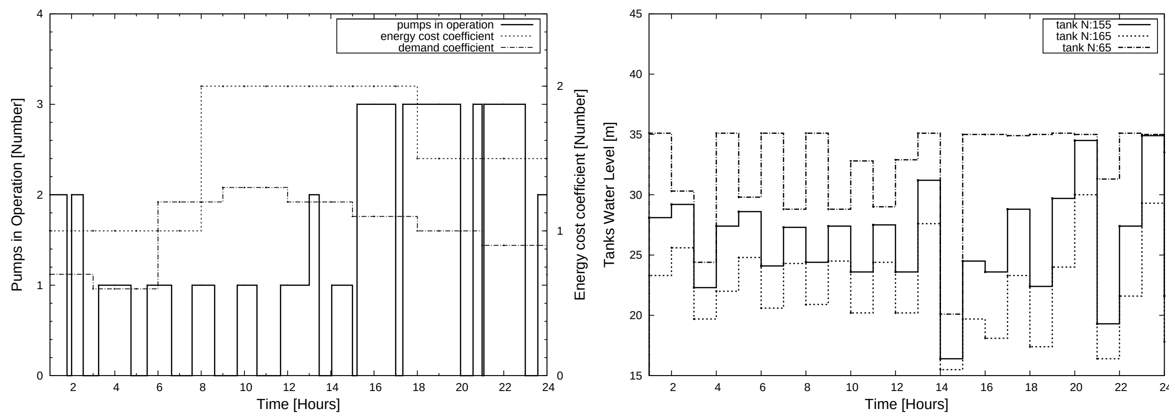


Figure 4.47.: Case study $O2$: Obtained pump operation schedule and tank water level for the final solution

Model Validation - Although the identified pumping schedules for the primal (Figure 4.45) and the final solution (Figure 4.47) are feasible in terms of constraints satisfaction, coincide with logical deduction and seem to be reasonable, their global optimality is hard to prove. Even more due to the lack of similar studies it can not be proved whether the results achieve better or worse from the others. Nevertheless the Farmani et al. (2005) study of the same system, identified the same tank position as the optimal one. Since this study aims at the generation of the payoff matrices among system investment costs and resilience[21] as the measure of the system reliability, different tank sizes have been identified as the optimal for the different reliability levels. Furthermore, Farmani et al. (2005) simultaneously considers design and operation analysis of water supply systems, optimizing pipe diameters and rehabilitation decisions, tank location and sizes and pump operation schedules for the multiple loading conditions at the same time. Obviously the presented results need further verification on other case studies. Nevertheless, it can be stated that the initial results seems to be logical and satisfactory.

Model Sensitivity - The progress of the Simulated Annealing algorithm during the identification of the primal minimum costs pumping schedule is presented at the left graph in Figure 4.48. Two phenomena are important to notice. First, the gradual reduction in the acceptable range for the solutions that is typical for the Simulated Annealing algorithm and

[21]the measure of the more power than required at each node

comes as a consequence of the ”temperature cooling” or reduction of the probability for the acceptance of the worse solutions than on the previous temperature level. And second, the lack of some systematic in the identification of the minimum solution. Due to the random creation of the new solutions, the occurrence of the optimal solutions is truly random.

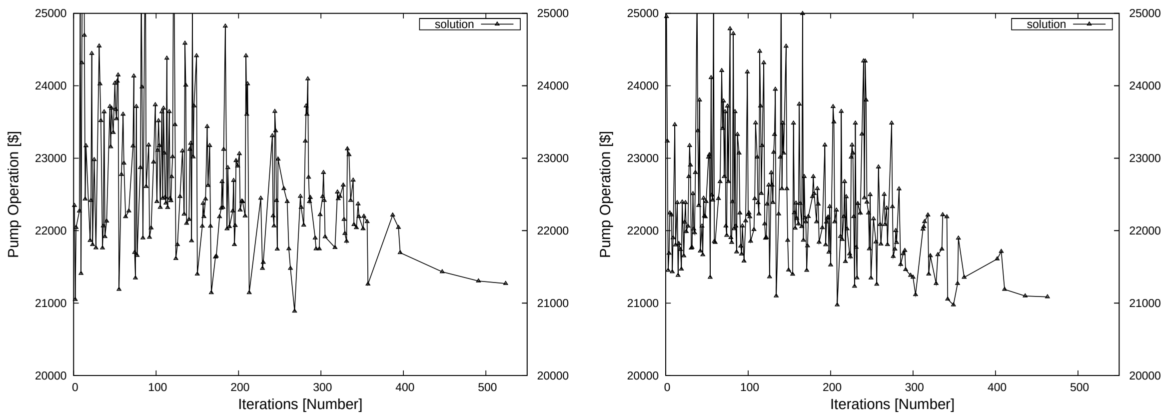


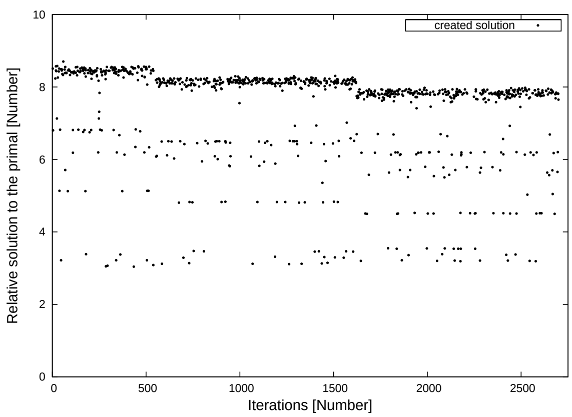
Figure 4.48.: Case study $O1$: Progress of the optimisation for random and weighted neighbourhood function

The way to increase the probability of identification of the global optima is to improve the way of creation of the new solutions, or so called ”neighbourhood function”. With this aim, the inverse value of the daily energy price coefficient (Figure 4.38) has been incorporated in the creation of the new pumping schedules as a weighting factor that increase the probabilities of selecting of ”on” states for the periods of lower energy prices and ”off” states for the periods of higher energy prices. In comparison with the purely random Simulated Annealing algorithm that needed 524 iterations to identify the minimum cost pumping schedule (the left graph in Figure 4.48), the Simulated Annealing algorithm with corrected ”neighbourhood function” needed 463 iterations to identify the same optimal solution (the right graph in Figure 4.48). As previously stated, the ”cooling schedule” is the next model parameter that determine the sensitivity of the presented procedure and has to be accommodated for each specific problem.

Beside optimisation parameters, the model is obviously extremely sensitive to the system parameters such as pump’s characteristics, tank’s operational levels, operational rules for pumps or valves, and many other. These are always too case specific to bring some general conclusions about their influence on the model. Furthermore, one could logically assume that the optimisation procedure is also highly dependent on the adopted time step for the extended time simulation analysis. Nevertheless, this is not the case since the most of the pump operations in real life systems are controlled by pressures on some predefined network points, which is a constraint that override the randomly created pumping schedules and increase the feasibility of the created pumping alternatives.

Model Efficiency - The efficiency of the Simulated Annealing algorithm can be improved by accommodating the parameters of the algorithm (e.g. temperature decrease, allowed number of iterations at each temperature level, etc.) or by the improvement of the ”neighbourhood function”, such as in Figure 4.48. Nevertheless, as it can be seen on both graphs in Figure 4.48, the creation of the optimum pumping schedules is a random function. The best solution may be created already at the beginning of the algorithm and not repeated later. This stress once more non existence of the theoretical prove of the global optimality of the identified optimal solution.

Looking at the progress of the final minimum cost optimisation procedure (the left picture in Figure 4.49) a typical step wise improvements of the Branch and Bound algorithm can be noticed. They are created by the step wise increases in tank and pump investment cost reduction during the examination of different combinations of the system configurations. The efficiency of the procedure mainly depends on its ability to reduce the examination of the combinations of tanks positions and sizes that can not yield optimal solutions. This process, so called "fathoming", depends on the order of the combinations that are to be examined and in order to make it more efficient all available information about the structure of the system and cost dependencies have to be included. This is a very system specific task.



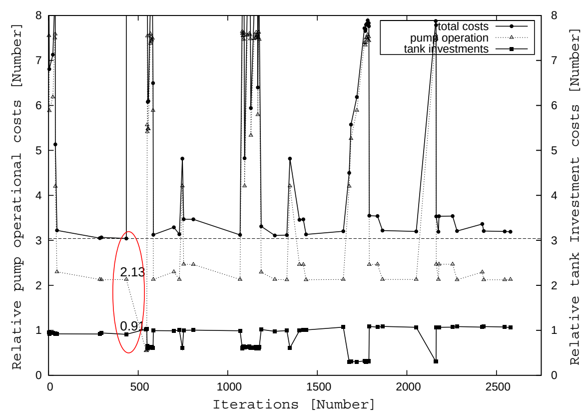


Figure 4.49.: Case study $O2$: Progress of the optimisation for the final solution

On the right picture in Figure 4.49, the progress of the identification of the final optimal pumping schedules for each new tanks combination is presented. The inverse dependence between pump operation costs and the tank investment costs is again to notice. For every solution with low investment costs only expensive pumping schedules could be found. The values of the operation and investment costs in Figure 4.49 are referenced to the primal solution and represent the ratios to the corresponding values of primal solution. The finally identified optimum solution is circled. It can be noticed that the final solution has 0.91 times lower investment costs than the primal one. This is achieved by the reduction in the tank area size. This reduction of the storage volume constraints the identification of the optimum pumping schedules and the finally identified optimum schedule is 2.13 times worse then the primal one. Nevertheless, due to the fact that the investment costs have much larger scale, the total costs of the final solution ($2.13 * 2393.63 + 0.91 * 262390.50 = 243873.24$ \$) are for about 8 % lower then of the primal solution (264784.12 \$). The ability of the procedure to make such trade-offs is its most important characteristics.

5. Conclusions and Outlook

Keeping in mind that the main objective of this study was to develop a modelling methodology for multi-objective analysis and optimisation of planning, design and operation of water supply systems, the suggested methodological concepts just as well as their implementations in the three computer models are discussed in this chapter. The main advantages and some of identified disadvantages with respect to the achievement of the stated research objectives are summarized. Based on this, some needed further improvements and possible new research activities are given.

5.1. Methodology Development

An attempt to further approach the integrative consideration of natural environment and human built-in water supply systems has been made by developing the methodology that aims at the multi-objective and risk-based decision support in planning, design and operation of water supply systems. In order to develop an systematic, integrative, transparent, and above all easily applicable methodology, the following issues have been treated:

Joint analysis of technical, environmental, economic and social aspects of water supply systems - When developing a new or analysing an existing water supply system it is of crucial importance to encompass all different positive and negative impacts it may has on a wide range of environmental, economic or social values. In order to enable the integrative consideration of impacts that have different units, times and scales, they are approximated with single-variable functions of some system parameter such as flow, capacity, water withdrawal etc.. Although such simplified impacts quantification adds to the uncertainty of the methodology, its has been argued that: a) the integration of various impacts is of much greater importance that a more accurate analysis of only one, b) the accuracy of the impact functions may be accommodated to the purpose of an analysis and available time, money and other resources, and c) the here introduced uncertainty has a same order of magnitude as most other input parameters of the analysis such as predicted water demands, interest rates, hydraulic characteristics, etc.. Furthermore, the methodology has been accommodated to cover a broad range of possible impact relations and to deal with various possible forms of impact functions (i.e linear, convex, concave and step-wise). In comparison with already existing methodologies for the optimisation of water supply systems this represents a significant improvement.

Integration of interests and objectives of different stakeholders and decision makers already in the formulation of alternative solutions - Since the creation and evaluation of a project or set of actions is highly dependent on preferences (utilities) of decision makers toward individual objectives or criteria, these preferences are included in the systems' analysis. The developed methodology adopts the Multiple Criteria Decision Analysis approach in which the selection of the optimal solution is done by a trade-off among different identified optimal solutions that correspond to different combinations of decision makers' preferences. It has been stated that the incorporation of the preferences already in the formulation of alternative solutions: a) significantly influences the direction of the optimisation search and improve efficiency of the optimisation algorithm, b) advance the development of the alternatives that are better suited to the decision makers' preferences increasing the chances for the selection of these solutions, and c) enable ease identification of the broad range of alternative options simply by changing the preferences (weights) toward different objectives. If known that in most decision support systems first develop optimal alternatives and then evaluate them according to the preferences of the decision makers, the advantages of the integration of decision makers's preferences already in the alternatives' development phase are more then obvious.

Handling of very complex problems with simple enough and easily understandable methods - Just as well as it is important to develop a methodology that is able to encompass the very sophisticated structure and function of water supply systems and the complex, multi-objective and multi-preference nature of water supply management problems, it is equally important that developed methodology stays simple and understandable enough to be applicable by water supply practitioners. In order to provide for an understandable representation of the water supply systems structure, the Network Concept from the Graph Theory has been used. The Minimum Cost Flow Network optimisation problem that is accommodated to handle multiple criteria and fixed and variable impacts at the same time, has been used to formulate the objectives of the analysis. Finally, a very robust optimisation technique (i.e. the Simulated Annealing) has been used to solve this non-linear and non-convex optimisation problem defined on conforming flow paths. The dependence of network flow from other network characteristics allowed for the ease application of the same problem formulation in the optimisation of different parameters such as system layout, sources' withdrawals, pipe diameters, tank capacities, pump schedules, etc.. In order to improve the ability of the procedure to deal with discrete problem and create and question a wide range of possible system configurations, the Simulated Annealing is embedded within the Branch and Bound procedure. The combination of these techniques applied on network problems, attained very similar results and proved to be slightly more efficient than most of the other often used heuristic techniques such as Genetic Algorithm.

Incorporation of robustness, flexibility and reliability considerations in the management of water supply systems - Robust, flexible and reliable planning, design and operation of water supply systems are new focus areas of the modern management of water supply systems. In addition to the implicit consideration of the systems robustness and flexibility by incorporation of multiple objectives and criteria in the development of management alternatives, the reliability issue has been explicitly addressed. The methodology has been accommodated to deal with the component failure analysis. An advanced method for the addition of spare capacities to the network systems (i.e. the Path Restoration method of Iraschko et al. 1998; Iraschko and Grover 2000) has been applied to identify the least cost system configuration that can provide for the functioning of a system in a case of failure of some component. The same method can be used to address any other deterministically defined stress condition, providing for a powerful optimisation tool especially useful in the design analysis of water supply systems.

Incorporation of uncertainty considerations, users' expectations and their risk tolerance into the evaluation of alternative management options - In addition to the deterministically defined stress conditions, the consideration of uncertainty and variability issues is just as equally important for flexible and reliable water supply systems planning, design and operation. An advanced sampling based technique (i.e. the Latin Hypercube Sampling of Iman and Shortencarier 1984) has been employed to efficiently create samples of uncertain or variable parameters that have some predefined probabilistic characteristic and can be used for the testing of the systems performance and the calculation of their reliability in terms of the probability of non-failure. The evaluation of the calculated reliability then is subjective to the users' expectations from the systems or the decision makers' risk perception toward some system failure. The incorporation of these two aspects in the methodology aligns with new engineering trends toward the substitution of fixed engineering standards and codes of practice with more risk-oriented approaches for management of water supply systems.

Transparent and easily applicable decision support for the integrative management of water supply systems - Finally, the transparency of the methodology is one of the basic prerequisites for its acceptance in praxis. An effort has been made to provide for the complete transparency of the methodology starting from the definition of objectives, adoption of criteria and selection of decision variables, through the approximation of impact functions, control of the optimisation procedure and creation of Pareto-optimal alternatives to the integration of decision makers' preference, their risk perception and evaluation of the parameters' uncertainty and reliability of water supply systems. This has been seen as a good way to promote greater involvement and participation of the decision makers since they are not just involved in the selection of some predefined alternatives but the alternatives are accommodated to their perception of the needed system performance and reliability. Furthermore, the multi-objectivity and risk-orientation of the analysis enable for the risk prone decision makers to sacrifice on some objectives (e.g. system performances in terms of delivered pressures) in order to achieve better on some another (e.g. savings in costs). This should add to the identification of sustainable development options that may improve the long term management of water supply systems.

5.2. Models Development

In order to enable easier use and application of the developed methodology, three computer model have been developed. Since the main aim and the purpose of the analysis differ for each management stage, the general methodological concepts have been accommodated to address the specific issues of planning, design and operation of water supply systems and have been implemented into corresponding models.

Planning Model - The fundamental questions of water supply planning studies such as the selection of the natural resources for human use, determination of the extent of water extraction and the identification of the most optimal water distribution can be addressed with the developed planning model. Furthermore, the multi-objective and multi-preference problem formulation provides for the identification of a wide range of solutions that are optimal for different combinations of preferences toward individual objectives. For example the single-objective solutions serve for the identification of the effects of the advancement of only economic, environmental or social criteria. In contrast, the multi-objective solutions offer a wide range of Pareto-optimal alternatives for the negotiation and trade-off among decision makers in order to identify the best compromise solution. It has been proved that the model is: a) able to deal with multiple objectives and criteria in a systematic way, b) able to encompass multiple preferences toward different objectives and to identifying a full range of valid solutions and c) has slightly better efficiency then some other approximation algorithms due to the more precise representation of the systems. Nevertheless, the main deficiencies of the model are: a) a very simplified representation of the effects and consequence of the systems with the impact functions and b) non-existence of the theoretical proof for the optimality of the identified solutions.

Design Model - The multi-objectivity of the water supply design problem reflects not so much in the integration of different systems impacts, but much more in the need to integrate different categories of importance into one analysis. Beside technical and economic issues, the incorporation of uncertainty, risk and reliability plays a prevailing role for the determination of optimal systems sizes and capacities. Therefore the design analysis is composed of two steps. The design model first identifies the minimum cost system configuration that satisfies all given constraints. The validity of this single-objective solution has been proved on two theoretical case studies. Furthermore, the model showed a very good efficiency in comparison with some other very often used approaches such as Genetic Algorithm. Then, the performance of the systems is assessed by implementing the component failure analysis and the parameters' uncertainty analysis. The combination of these two provided for the easily manageable and clearly understandable assessment of the systems reliability in terms of the probability of not-failure. The decision makers' risk acceptance level present the basis for the selection of the optimal design option by trading-off between the system's costs and its reliability. It has been proved that the model is able to: a) deal with complex, discrete, NP-hard problem of the selection of the minimum cost water supply network configuration, b) identify just as good and valid solutions as the other models reported in the literature with approximately the same or even better efficiency of the algorithm, c) address the reliability issue with the combination of the deterministically defined component failure analysis and the stochastically based performance failure analysis for some uncertain or variable parameters and d) allow for the risk-based selection among system performances and reliability on one side and economic costs on the other. Some important limitations of the model are: a) simplified consideration of the pressure distribution within the optimisation model, b) backward going approach for the assessment of the system reliability and addition of an external network solver for more precise calculation of the system performances, and c) simplified characterisation of the mutual dependencies of the uncertain and variable parameters in form of correlation matrices.

Operation Model - The identification of possible trade-off between investment costs in water storage facilities and operation costs of water pumping stations is adopted as the main problem to deal with. Although both mentioned objectives are expressed in the same terms, the addition of time dimension to the optimisation problem significantly adds upon its complexity. Furthermore instead of water flow as the one main decision variable like in the planning and the design problem, the tanks sizes and pumping schedules now represent two distinct decision variables. Both variables are discrete and the pumping schedules have an additional dimension since they are distributed in time. The addition of the time dimension significantly adds up on the model complexity since the flow vector is not any more a stationary value but instead the set of, in time ordered, flow vectors. Furthermore, a network solver of Gessler et al. (1985) had to be coupled with the optimisation model for the calculation of network flows, pressures and tank levels during the extended period simulation. Nevertheless, the model managed to identify reasonable pumping schedules and tank configurations for the two applied case studies. Unfortunately the exact validity of the results could not be proved due to the lack of similar studies. It has been stated that the model: a) has a possibility to model various components such as pressure reducing valves, check valve, booster stations, etc. and different sort of operation rules, and b) is able to deal with a complex problem of simultaneous selection of the minimum cost tanks positions and sizes and the identification of the optimal pumping schedules. Still the main limitations for its application in operation of water supply systems are the facts that: a) it focuses only on the trade-off among tank investment and pump operation costs and b) the optimality of the produced solutions can not be theoretically proved.

5.3. Outlook

From the presented study it can be concluded that the use of the network concept provides a very good conceptual representation and increases the number of additional information that can be assessed in structural or capacity analysis of water supply systems. Furthermore, it improved the efficiency of the optimisation algorithms and added to the applicability of the developed methodology. As far as the integration of various objectives into one modelling environment is concerned, the disadvantages and difficulties in the development and validation of the impact functions have been recognized. Nevertheless such an approach enable integrative analysis of different economic, environmental and social issues and is adopted as accurate enough. The implementation of the decision makers' preferences in the development of the optimal alternatives has been recognised here as more important. In addition, the evaluation and selection of the optimal alternatives has been accommodated to the risk perception of the decision makers in order to provide for their greater participation and development of sustainable development options. Finally, the suggested methods for dealing with uncertainty and reliability issues are deliberately chosen to be as simple as possible in order to enable for their ease accommodation in different water supply analyses. Nevertheless, they provide for systematic and transparent incorporation of these issues in planing,design and operation of water supply systems.

Based on the results of the models implementation, it can be stated that the developed methodology provides for the achievement of the stated objectives. Nevertheless, the more detailed testing and validation of the models is necessary. Application of the models on some real-life cases would be especially beneficial as well as the confrontation of the produced results with the expert's knowledge from the field. Furthermore, although based on the same methodology the individual models are still functioning completely isolated. An integration of the developed planning, design and operation model into one decision support system for water supply systems management would not only improve the data exchange among models, but also significantly add upon their usefulness and user-friendliness. In addition, many additional options (such as new objectives and decision variables) could be included in the models and especially in the operation model. Similarly, the rehabilitation stage of the water supply management could be developed on the same methodological concepts but as a separate model. Finally the applied methods can be exchanged with some others just as long as this improve the efficiency, applicability or transparency of the methodology. The selection of the individual methods for the solution of the network problem, multi-objective optimisation, or uncertainty, risk and reliability assessment is quite irrelevant in comparison to the importance of integration of these issues in one analysis.

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A. Appendix

A.1. Environmental Impacts of Water Supply Projects

Issue	Possible Causes	Typical Effects	Impact Assessment	Mitigation
Dust during construction	Entrainment of dust from roads, stockpiles	Public nuisance	Air quality modeling	Dampen roads, cover stockpiles
Fogs and mists	Increased water vapor in atmosphere	Increased incidence of fogs and mists	Water balance calculation	

Table A.1.: Impacts of water supply systems on air quality [source: CIRIA, 1994]

Issue	Possible Causes	Typical Effects	Impact Assessment	Mitigation
Lower groundwater levels	Over-pumping	Loss of wetlands springs, river flows	Hydrogeological studies	Limit,redistribute abstraction
Change fluvial regime	River intake	Reduction of river flows (min. flows)	Hydrological studies	Operating rules, better construction
Water-logging	Reservoir	Local rise in water table	Hydrogeological studies	Bed lining, level control
Downstream water quality	Lower river flows	Higher concentrat. of pollutants	Water quality studies	Compliance with flow regime
Reservoir water quality	Nutrient build up, algal growth	Eutrophication, downstream pollut.	Water quality studies	Nutrient reduction Destratification

Table A.2.: Impacts of water supply systems on water quantity and quality [source: CIRIA, 1994]

Issue	Possible Causes	Typical Effects	Impact Assessment	Mitigation
Loss of mineral resources	Inundation, building on mineral land	Sterilization of sand and soil deposits	Soil studies	Avoid mineraly valuable sites
Slope stability	Steep slope, high pore water pressure	Slope failures	Geotechnical studies	Site investigation dam design
Soil erosion	Rains during excavation	Loss of soil, higher deposition rates	Hydrological studies	Runoff control, soil protection
Seizmology	Increased pressure and faults lubrication	Increased incidence of earthquakes	Geological studies	Avoid tectonically unstable areas

Table A.3.: Impacts of water supply systems on land [source: CIRIA, 1994]

Issue	Possible Causes	Typical Effects	Impact Assessment	Mitigation
Permanent inundation	Accumulation	Loss of habitats	Ecological studies	
Wetlands degradation	Groundwater & river flow regime changes	Loss of flora & fauna accumul. of nutrients	Ecological studies	Maintenance of natural regime
Rivers ecology changes	River abstraction, physical barriers	loss of species number & diversity	River habitat studies	Maintenance of sufficient flows
Estuaries degradation	changes in river quantity & quality	Changes in food chain, species distribution	River habitat studies	Maintenance of minimum flows
New habitats	Creation of new water bodies	Attract wildlife, used for fisheries	Ecological studies	Consider wildlife

Table A.4.: Impacts of water supply systems on natural habitats [source: CIRIA, 1994]

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Verzeichnis der Mitteilungshefte

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