CHAPTER FOUR

CHAPTER 4

INDEX

TABLES

ABBREVIATIONS AND ACRONYMS

%	-	percentage
ABC	-	Activity Based Costing
AC	-	Asbestos Cement
ASCE	-	American Society of Civil Engineers
AWWA	-	American Water Works Association
BS	-	British Standard
BSI	-	British Standards Institute
C.I.	-	(grey) Cast Iron
CATT	-	controlled air transient technology
CC	-	City Centre
CM	-	Cement Mortar
CO_2	-	Carbon dioxide
DI	-	Ductile (cast) Iron
DICL	-	Cement mortar lined DI Pipe
DIN	-	Deutsches Institut für Normung e. V. (German Standard)
DN	-	Nominal Diameter
DP	-	domestic point
EN	-	Euro Norm
EP	-	Epoxy Powder
FBE	-	Fusion Bonded Epoxy
FR	-	flat rate
GRP	-	Glass Reinforced Plastic
GS or GI	-	Galvanised (hot dipped zinc coated and lined) Steel
HC	-	High Cost Housing
HIG	-	High Income Group Housing
I	-	Class of non or only low aggressive soils
II	-	Class of aggressive soils
III	-	Class of highly aggressive soils
ISO	-	International Standards Organisation
ISO	-	Insurance Service Office, USA (reference to Fire Fighting flows only)
ISO	-	International Standards Organisation
IWA	-	International Water Association
K_d	-	peak day factor
K_h	-	peak hour factor
LC	-	Low Cost Housing
LCA	-	Life Cycle Assessment
LIG-F	-	Low Income Group Housing - formal
LIG-I	-	Low Income Group Housing - informal
LIG-M	-	Low Income Group Housing - multiple household
MC	-	Municipality
MC	-	Medium Cost Housing
MDGs	-	Millennium Development Goals
MDP	-	Modified Proctor Density

MIG	-	Medium Income Group Housing
M-PBT	-	metered with progressive block tariff
MRS	-	Minimum Required Strength
M-UT	-	metered with uniform tariff
n	-	number of storeys in a building
NAWAPO	-	National Water Policy
NDC	-	National Development Corporation
NE	-	north east
NGO	-	Non Governmental Organisation
NPSH	-	net positive suction head
NRW	-	Non Revenue Water
NRWSSP	-	National Rural Water Supply and Sanitation Programme
O&M	-	Operation and Maintenance
P	-	Population
PEA	-	Allowable Site Test Pressure
PE_{comp}	-	PE pipes with compressed joints
PEHD	-	Polyethylene Pipe made from PE_{100}.
PES	-	Polyethylene Sheeting
PE_{weld}	-	PE pipes with electro-fusion welded joints
PFA	-	Allowable Operating Pressure
PN	-	Nominal Pressure
PN	-	Nominal Pressure
PP	-	Polypropylene
pr	-	provisional
PRV	-	pressure reducing valve
PUR	-	Polyurethane
PVCu	-	unplasticised Poly Vinyl Chloride
Q	-	total Quantity of water
q	-	average rate of daily consumption
Q_{da}	-	Average daily demand
q_{dim}	-	dimensioning flow
Q_{dmax}	-	Maximum daily demand
q_f	-	fire fighting demand
Q_{hmax}	-	Peak hour demand
RL	-	Real Losses
RQI	-	Ride Quality Index
RSC	-	Rural Service Centre
RW	-	Revenue Water
S	-	Pipe Series
SABS	-	South African Bureau of Standards
SDR	-	Standard Dimension Ratio
SE	-	south east
SI	-	International System of Units
SIDO	-	Small Industries Development Organization
std	-	student
TBS	-	Tanzania Bureau of Standards
TC	-	Town Centre

UAC	-	Unbilled authorised consumption
UC	-	Urban Centre
UfW	-	(Unaccounted for Water) former term for Non Revenue Water
UK	-	United Kingdom
UKWIR	-	United Kingdom Water Industry Research
UNICEF	-	United Nations Childrens Fund
US$	-	United States Dollars
USA	-	United States of America
UV	-	ultra violet
VC	-	Village Centre
WC	-	water closet
WLC	-	Whole Life Costing
WRc	-	Water Research Council (UK)

UNITS OF MEASUREMENT

”	-	inch
Bar	-	One atmosphere
ca	-	capita
cm^2	-	square centimetre
d	-	day
ha	-	hectare
hrs	-	hours
kg	-	kilogram
km	-	kilometres
l	-	litres
l/d	-	litres per day
l/s	-	litres per second
lb	-	pound avoirdupois
m	-	metre
min	-	minutes
MJ	-	mega Joules
m/s	-	metres per second
m^3	-	cubic metres
mm	-	millimetre
^oC	-	degrees Celsius (centigrade)
P	-	pressure in Bars
sq	-	square

CHAPTER 4

DESIGN OF PIPED WATER SYSTEMS

4.1 GENERAL

4.1.1 Revisions to 1997 Edition

Water amenable to treatment to make it suitable for human use is a diminishing and ever more expensive resource to exploit. At the same time, the cost of conveyance by pipelines continues to rise as either directly (thermoplastic pipes) or indirectly (ferrous pipes) require oil.

Because of these changes, this Chapter of the Design Manual has gone through a major revision, the first for nearly ten years. However, and as far as practicable, the layout of the Chapter is kept the same as in the earlier version although some sections have been split into two or more to make for easier reading.

In this updated chapter, the increasing availability of computational software is recognised so that earlier examples of simple handheld calculator type computations are no longer presented except as examples. Instead design spreadsheets are introduced together with worked examples to facilitate their understanding. Where other hand-held calculator design is still found necessary, Designers should refer to the 1997 edition of this Design Manual or to textbooks for such information.

It recognises the need to move forward in the way we consider alternatives and cost pipelines from the simple approach of supply price to a ‘zero-failure’ approach to ‘total cost of ownership’ to a ‘whole-life’ consideration based on the concepts of Whole Life Costing (WLC) and Life Cycle Assessment (LCA).

It acknowledges the need to manage water supply and water demand through both measurement and control and through the use of pricing mechanisms in the supply of water that cover costs in order to bring about changes in both the way we use water and in the amounts we use.

It takes into account developments in recent years in Piping and Ferrous Pipe Coatings and Linings to help extend the useful working life of such pipes. It discusses and specifically refers to a number of International and National Standard Specifications that Designers are encouraged to consider and adopt as well as internationally recognised design procedures.

As a result of these revisions, differentiation in the economic life of different pipes has been introduced, and previous water demand projections have been re-considered even though information from local sources in this area is still sparse. Because of this, the figures now suggested are to be used as lower bound guidelines only and as and where Designers have better local pertinent information or feel that the figures are now too low they should feel free to adjust them. It suggests levels to be allowed for physical water losses in determining the total flows to be catered for.

To facilitate improved accuracy in future water demand and physical loss figures, Designers are encouraged to submit substantiated differences to the figures given here to the Design Section of the Ministry so that periodic addenda can be issued in the field of water demand and loss criteria.

When it comes to Whole Life Costing, there is currently insufficient information worldwide to make a complete application possible. However for the larger schemes in particular, the designer is requested to take account in his designs of the environmental aspects of Life Cycle Assessment that are already known as well as all aspects of Whole Life Costing that can be quantified as noted above.

4.1.2 Introduction

The revisions incorporated in this Chapter not only reflect the developments of the last ten years but make use of many additional sources of information ranging from International and National Standard Specifications and Design Manuals to relevant articles released on the world-wide-web.

Currently, per capita domestic demand is going through change as charges are increased to become more market oriented, consumer metering is being introduced, and other demand management measures are being introduced. The reductions in per capita demand that these changes should cause can only be approximated to at present and Designers should carefully evaluate the figures now suggested and not simply adopt them without due consideration to their particular circumstances.

However, population and demand projections and other calculations presented here are based on what can be considered as local best practice and different sources of information gathered primarily in different regions of the country. In arriving at consumption rates and design criteria, factors such as water availability, the practical difficulties in abstracting it, charges and tariff structures, and the continued financial limitations on development capital in particular have been considered and the alternatives considered best for Tanzanian conditions in the present context have been suggested.

In an effort to ensure that this chapter can largely stand alone, a certain amount of repetition with other chapters is inevitable. However, wherever deemed practicable, this has been avoided by cross-referencing.

4.2 PIPE SELECTION

4.2.1 Alternative Methods of Approach

The ways in which pipes can be selected include:

- Supply-only cost
- Zero-failures cost
- Total cost of ownership, and
- Whole-life cost

A **supply-only** cost is no longer suitable even for the very smallest of schemes as it fails to recognise even the most basic of requirements of water supply security and all too often results in short working life and early failure of a water supply system. Moreover it is a very inefficient way of utilising scarce development capital.

Whilst a **zero-failures** approach has its limitations, it may still be used for smaller schemes where more detailed considerations are considered to be impracticable providing that this is clearly stated and has the Clients formal approval.

For larger schemes however, but as a minimum requirement, a **total-cost-of-ownership** approach may be adopted until such time as **whole-life costing** becomes more widespread in the water industry and the information necessary for this is more readily available than at present.

However, in recognition of the worldwide effects of climate change, it is recommended that even then, designers should consider both the embodied energy requirements of each pipewall material being considered as well as the other environmental consequences in manufacture, use and eventual disposal. A statement on the consideration given to climate change and other environmental impact of proposed pipes should be included in every design report.

These different approaches are discussed below.

4.2.2 A Supply-only Cost Approach

This was the approach adopted for many designs in the past with the supply cost often being based on historic or static rather than current or dynamic information and different pipe materials being pre-selected by Designers on the basis of diameter.

This tended to result in pre-selection of PEHD for the smallest of diameters (say nominal diameter DN15 to DN50), PVCu for the next diameter range (say DN65to DND160), D.I. for medium to large diameters (say DN200 to DN400) and Steel for larger diameters.

Such an approach is no longer acceptable. Nor is any final pre-selection by the designer because pipe prices in particular are continually fluctuating due to the instability in the international price of oil.

4.2.3 The Zero-Failures Approach

The zero-failures approach to pipe selection takes into account the total cost of supply and installation, and is dynamic in that the costs at the time of implementation are those that govern.

It avoids the need to consider the total ownership cost and allows the choice of material to be made through a much simpler comparison. By using an initial selection flow chart, it reduces the different types of pipe that need be considered and if properly applied brings the costs for operation, maintenance and repair to a similar order of magnitude, regardless of the alternative types of pipe that are then considered. Such costs can then be put aside in making the cost comparison.

However, a word of caution is required because even a zero-failures approach pre-supposes that firstly the design is professionally carried out and secondly that the specifications for installation are both sufficiently rigorous to make O&M and repair costs similar for all types of pipes being considered and that the specifications are strictly adhered to, especially in the areas of maintaining pipe minimum cover, use of suitable embedment materials and compaction. Regrettably, this is not always the case and as a result it is often pipe manufacturers that are blamed for failures where the fault lies with the entities that supervised and carried out the installation. This is particularly so for thermoplastic pipes, less so for steel pipes and least problematic for D.I. pipes.

The flow chart to be used initially where a zero-failures approach is adopted is included in Section 4.14.3.3 of this chapter.

Designers are however encouraged whenever possible to consider the broader aspects of the environmental and social implications of their designs from “cradle to grave” and even beyond when considering the materials available. The environmental consequences of today’s decisions on future generations should no longer be ignored. The materials of today will become the waste of tomorrow and ignorance of the consequences of our decisions is no longer an acceptable excuse for a professional design engineer.

4.2.4 A Total-Cost-of-Ownership Approach

This is the real economic cost of a pipeline and needs to take into account the supply, installation, operation & maintenance, and repair costs of the pipeline during its economic working life. Any resale value of the asset at the end of its working life should also be considered although this is insignificant or zero in most cases.

It requires a more rigorous approach to design than with a zero-failures approach but stops short of considering the broader social and environmental consequences not already required of manufacturers and hence built into the supply price.

For urban water supplies and the larger rural water supply schemes it is the approach which should be adopted.

Where a 'force-account' installation procedure is being adopted, it is however still a requirement that the designer takes installation and subsequent costs into consideration and indicates cost 'weighting' factors based on his best estimates of these to be used when evaluating different supply prices.

The method of determining and calculated values of such weighting factors need to be clearly indicated in supply-only bid documents so as to eliminate grounds for subsequent complaint from different pipe suppliers.

4.2.5 Whole Life Costing and Life Cycle Assessment

Whole Life Costing is a term that describes the costing of various aspects of sustainability in the design, manufacture, installation, operation & maintenance, repair, and eventual decommissioning and where appropriate re-use of a built asset or its properties. It entails achieving compromise and synergy between three different sets of costs and values, namely those of economics, environmental, and social. These may be defined as:

- **Economics** is focused on economically sound sustainability, is growth-oriented and safeguards the opportunities of future generations. This is usually determined using a traditional costing method involving the quantification of direct costs and indirect costs. There are however moves with the international water industry and elsewhere to Activity Based Costing (ABC) which considers costs under three headings namely: - Direct Costs; Activity related indirect Costs and Other indirect Costs. Because ABC is in its infancy in the water sector, a traditional costing method is discussed here.
- **Environmental** is focussed on environmental aspects such as pollution, waste, and CO_2 emissions. Environmental Value is maximised when environmental pressures are minimised to the level of the carrying capacity of ecological systems while using natural resources effectively and safeguarding natural capital and its productivity. This requires a life-cycle assessment.
- **Social** is focussed on social aspects such as social security and social equity. Occupational health and labour market relations are taken into account. 'Social' may be referred to as 'societal' covering a wider scope of social, cultural, ethical, and juridical impacts. This also requires a life-cycle assessment.

Hence a whole life water pipeline design and selection, requires the consideration of all costs and values of a pipeline in its design and selection. It requires the consideration and as far as practicable, the cost quantification of numerous factors, some of which are frequently changing due to external factors and in particular due to such things as international demand for the raw materials involved and oil prices. In gravity pipeline systems, the initial investment cost predominates. In pumping systems, power costs over the whole life of the system are also very significant.

Whole life pipeline design involves taking into account such factors as: -

Economic Cost (total-cost-of-ownership costing):

Design:

- Design working life.
- Determination of carrying capacity and nominal hydraulic pressure based on: -
    - Design flow (including an allowance for technical losses),
    - The internal diameter, and
    - Roughness (friction) of the lining.
    - And in pumping systems the capital and operational power cost associated with the selected diameter.
- Wall thickness under the prevailing site conditions, including taking into consideration such things as:
    - Ambient and water temperatures,
    - Loss of ability to withstand pressure over time, and
    - Damage risk due to such things as: -
        - Subsequent laying of other services nearby,
        - Damage during tapping-in to add consumer or other connections,
        - Vandalism,
        - Vehicular impact,
        - Risk of vacuum conditions, and
        - Likelihood, frequency and magnitude of surge.
- Coating and lining of ferrous pipes based on soil type and chemistry of the water to be conveyed.
- Pipe joint and connection alternatives.

Supply (direct costs):

- The ex-works supply cost which must take into account: -
    - Raw material quality control and cost,
    - Manufacturing quality control, including the general and specific tests required,
    - (Thermoplastic) Storage requirements including those of temperature and shade,
    - (Ferrous) Coatings and linings necessary to meet working life requirement,
- Delivery from country of origin to storage site in country of use including all insurances.
- The delivery to site cost which must take into account: -
    - Damage risk, especially to coatings and linings and pipe ends,
    - Damage risk during loading and off-loading,
    - Protection during on-site storage and laying, and
    - Cost of checking for damage and making good at trench side.

Installation (direct costs):

- The lay cost which includes: -
    - Ensuring minimum cover and hence excavation volume and amount of rock to be excavated,
    - Embedment and backfill materials, their bringing to site and compaction requirements,
    - Disposal of surplus or waste material. and
    - Damage to other infrastructural elements such as cables, sewers, water pipes, roads and drains during excavation and their making good.

Operation and Maintenance (indirect costs):

- Routine operation and maintenance
- Cost of subsequently installing additional consumer connections and inserting new fittings,
- Cost of repairing leaks and bursts
  - Damage caused by vandalism and vehicles, etc., and
  - Other damage
- Value of water lost through leakage and bursts.

ENVIRONMENTAL COST (LIFE-CYCLE ASSESSMENT):

- Environmental considerations including those in the: -
  - Manufacturing of raw materials, including pollution and energy consumption
  - Manufacturing of pipes and fittings, including pollution and energy consumption
  - Damage caused to other property through leakage and bursts of installed pipeline , and
  - De-commissioning either by abandoning or removal and disposal after life expiry.

SOCIAL COST (LIFE-CYCLE ASSESSMENT):

- If it is a Government's policy to provide a local preference in order to promote employment and industrialisation and this is acceptable to the Financier, this is a social cost to be included.
- Disruption to the public both when first laying, when repairing and when augmenting or replacing at end of working life if different working lives are considered.
- Social costs of rationing, low pressures, unplanned interruptions, poor water quality and health implications.
- Complaints from Consumers and the Public at large.

Pipe Selection

As noted in the preceding sections, in the past it has often been the Supply price at the time of Design which has tended to govern pipe selection and all too often Designers have done no more than consider nominal diameter instead of the actual internal diameter for the hydraulic calculations and simply selected a pipe diameter and pressure class solely on the basis of carrying capacity, the required maximum hydraulic pressure and a knowledge of recent ex-works or delivered supply costs. In the case of ferrous pipes, often a cursory evaluation of soil and water conditions has led to the selection of what later has been shown to be an inappropriate coating and/or lining.

From the above, it is clear that such a design approach is at best simplistic and at worst non-professional when making a comparison and hence a decision on pipe selection. It is woefully incomplete if the most economic solution is to be achieved. It is totally inadequate if whole life cost is to be considered and whole life value is to be achieved.

Whilst all direct 'economic' items can and should be costed, and best estimates made of 'indirect' costs, environmental [< >] and social values are much more difficult if not impossible to

[< >] For example raw materials and many manufactured goods are produced outside the country so that any adverse effects may not be felt by the inhabitants at all, e.g. asbestos cement mining and pipe manufacture, or felt only indirectly or over a long period, e.g. PVC raw material manufacture. However, eventual disposal of the finished product usually will be in the country of use and that is where problems can arise, e.g. burning at normal temperatures, deliberate on building sites or otherwise in landfill, of PVC products.

quantify, and therefore require a value judgement. Hence the need for Life-Cycle Assessments (LCA).

Having said that, LCA's are still in their infancy but with the publication of ISO 14040 are gaining rapid ground and should no longer be ignored where consideration is possible.

Design or Working Life of System Components

As yet there has been only a limited application of Life-Cycle Assessment when it comes to optimising pipe replacement frequencies. However in 2004, one useful study in this regard for urban areas at least was reported where a life-cycle energy analysis was conducted for a major metropolitan area in the USA < >. Looking at four scenarios between 10 years and 100 years, this showed that a 50 year pipe replacement period was optimum in that instance.

As disruption and energy cost associated with pipeline replacement in a Tanzanian environment is unlikely to be as high, it is suggested that for urban areas a 40 year design working life should be adopted and that for rural areas a figure of not less than 30 years be considered.

The working life of a pipeline should not however be confused with the design period of a project or project phase which is usually much shorter.

4.3 Definitions and Other Background Information

4.3.1 General

Various definitions and other background information is presented this section to guide design engineers, who are requested also to read Chapter 1, section 1 in this regard.

4.3.2 The Registered Village

The basic formal unit in Tanzania is the Village. The history of the village level approach in Tanzania goes back nearly 40 years to the birth of Ujumaa when neighbouring households were grouped together in community units of at least 250 households, amongst other things so as to be able to provide them with services such as health, education and water.

In 2002 Tanzania reportedly had some 10,832 registered villages < > both rural and urban. A village forms the unit of measurement in the National Population Census and is in most cases a readily identifiable community, with its boundaries described. In the vast majority of instances a village has less than 5,000 inhabitants. Villages are usually sub-divided further into several sub-villages or hamlets.

Nevertheless, the Village community is usually a homogeneous entity, especially in rural areas and therefore forms the logical starting point for NAWAPO, as the unit for supplying and managing water schemes at the lowest appropriate level.

The village is the focal unit for the Water Sector Development Program (WSDP) commencing in 2007/08 and in rural areas in particular is the level which planners and Designers should first consider, particularly where protected springs and groundwater are feasible and cost effective options.

< > Life-Cycle Energy Analysis of a Water Distribution System. Yves R. Fillion, Heather L. MacLean, and Bryan W. Karney, Journal of Infrastructure Systems © ASCE, Sep. 2004

< > Land Issues and the National Development Strategy: The Tanzania Experience. A Paper prepared for the World Bank sponsored Regional Workshop on Land Issues in Africa, Kampala - Speke Resort, April 29 – May 02 , 2002,

4.3.3 Classification of Rural and Urban Centres

The following definitions for various population centres in an ascending order of size as given in the past by Ministry of Lands, Housing-Urban Development. They are presented as a guide to Designers.

(a) Village Centres (VC)

A settlement below the status of a rural service centre as defined below is called a village centre.

(b) Rural Service Centre. (RSC)

A settlement with a population below 5,000; having the facilities listed below is called a Rural Service Centre. Primary school, dispensary, police post, daily market, postal agency, agricultural marketing depot, seed and fertilizer. depots and agricultural credit services as well as tractor and plough hire services.

(c) Urban Centre (UC)

A settlement having a population above 5,000; ten percent of which are in non-agricultural occupations is called an Urban Centre. It has the facilities as the Rural Service Centre at an improved level in quality and quantity.

(d) Town Centre (TC)

An urban centre having a population of 15,000 and above, and 30% of the population in non-agricultural occupation is called a Town Centre. It has the following facilities: a water supply system, electricity services, a bank, a hospital and a secondary school.

(e) Municipality (MC)

A town centre with a population of 80,000 and above, and 60% of the population in non-agricultural employment is called a Municipality. Its facilities in addition to those of a Town Centre are: more than one bank, one of which must have an exchange control service. In addition to having the increased facilities of a town centre, a municipality has an organized transport system.

(f) City Centre (CC)

A municipality with a population of 200,000 and above, and 75% of its population in non-agricultural employment, is called a city. In addition of having the same facilities as a Municipality, a City may also have a university

4.3.4 Urban Housing Classification

Housing should never be classified by density but by a householder's income or such things as plot and area layout, building construction materials and floor area. Allowance must be made for households occupying multiple dwelling rented accommodations. Many high income dwellers occupy housing built in medium density areas whilst in rural and some peri-urban areas, low and medium income housing is constructed on relatively large plots. Therefore, housing is best classified as low income-informal, low income-formal, low income-multiple household, medium income or high income group housing and in urban areas may be defined for planning purposes as shown below: -

(a) Low Income Group Housing (LIG-I) - informal

This group is mainly squatter housing built of impermanent materials or a mix of impermanent and permanent materials and occurs in pockets within larger urban areas and on urban outskirts or in peri-urban areas. It is usually of very high density development. Dwellings rarely have a water supply and reliance is placed on distribution points (DPs) or on water cart vendors. Unimproved pit latrines are provided for sanitation.

(b) Low Income Group Housing (LIG-F) - formal

This group is found mainly on urban outskirts or in peri-urban areas but the building is of permanent materials. Generally of high density development. Dwellings, if connected to the water supply, are normally furnished with yard taps and very simple piping. Inside installations are rare and there is only an outside water point for dish and clothes washing. Pit latrines are provided for sanitation.

(c) Low Income Group Housing (LIG-M) – multiple household

This group is found mainly in pockets within larger urban areas, on urban outskirts or in peri-urban areas but the building is of permanent materials. Generally of high to very high density development. Such buildings provide rented accommodation, often single rooms, to single persons or married couples without children. Such buildings are normally provided with very simple piping, yard taps for dish and clothes washing, one or more shower cubicles and one or more WC facilities.

(d) Medium Income Group Housing (MIG)

Generally lower density development than LIG housing. Houses are normally furnished with at least internal piped to a kitchen with cold water for dish washing, and bottled gas or electricity for cooking. The dwelling also has a shower, and where there is a mains sewerage system or a septic tank, with a water closet (WC). An outside tap is provided for clothes washing.

(e) High Income Group Housing (HIG)

Generally lower density development than MIG housing, but not necessarily so. Houses are provided with internal piped multiple taps, cold and hot water systems, and electricity supply, bathrooms/showers, an internal arrangement for cloth and dish washing with WCs and mains sewerage or septic tanks facilities. Accommodation for one or more domestic / garden staff is often also on the property, this being provided with a cold water tap, shower and WC.

4.3.5 Economic Life of Pipelines

In the absence of more specific data, the information in the table on the following page gives the recommended economic lifetimes for pipelines: -

TABLE 4.1: RECOMMENDED ECONOMIC LIFE OF PIPELINES

	COMPONENT OR ASSET	ECONOMIC (DESIGN) LIFE IN YEARS (R=rural, U= urban)	ANNUAL MAINT-ENANCE COST AS % OF CAPITAL COST
Piping: -	Uncoated steel [not recommended]	Both: 7	
	GS/GI (hot dipped zinc coated steel) [Avoid if possible, otherwise ND less than 80 mm only]	Both: 10	
	FBE coated, zinc lined steel pipes (DN65 to DN25 only), for class I and II soils for class III soils	 Both 30 Both 20	
	PVCu, min. cover 900mm, and S1/S2 embedment	R 30/ U 20	
	PEHD, min. cover 900mm, and S1/S2 embedment	R 40/ U 30	
	PVCu, min. cover 1500mm, and S1/S2 embedment	U 30	
	PEHD, min. cover 1500mm, and S1/S2 embedment	U 40	
	D.I., min cover 600 mm, coated for Soil Type III conditions and suitably lined for quality of water conveyed	Both 40	
	Steel, min cover 600 mm, coated for Soil Type III conditions and suitably lined for quality of water conveyed	Both 40	
	Pre-stressed concrete trunk / primary mains [not recommended]	Both 30	
	GRP (rural trunk mains only) [not recommended]	R 30	
	AC [not to be used]	-	
For suggested Economic Lives of other system components see Chapter 8, section 8.4.2.			

4.4 PROJECT PLANNING

4.4.1 National Water Policy

All new and rehabilitation work in the water and sanitation sector should be in accordance with the National Water Policy. Please refer to Chapter 1, section 1 for more information.

4.4.2 Planning of Piped Rural Water Supply Schemes

Whilst rural, village level, water supplies based on a protected spring or groundwater are feasible in many parts of the country, there are areas and particularly on Mountain and Hillside slopes and where basement complex rocks are present that are not amenable to a discrete supply of water to an individual village.

In such circumstances a piped water supply providing water to a group of adjacent villages may be the only practicable solution.

However, and when planning such schemes, the village community should still be taken as the level at which the individual village must manage its supply and the feeder pipelines conveying water to the village should be designed in such a way that they do just that and are not used as combined feeder and distribution pipes.

Supply to and any detailed distribution within a village should then be through a limited number of bulk water meters (ideally one only) from the feeder main so that apportionment of water between villages can be monitored and control effected, and villages further from the source of supply can see that they are not being deprived of their rightful share. Wherever practicable a principle village storage tank should be provided immediately downstream of the bulk meter to enable the feeder main to be designed for as close to average day flow as possible and to allow for any onward supply distribution into the village to be based on a peak hour flow concept.

The group of rural villages in a piped supply scheme must between them, then take responsibility for the source and feeder piped system through the employment of a private entrepreneur or a board of trustees or other organisation found suitable[< >].

4.4.3 Planning of Piped Urban Water Supply Schemes

Except in such instances where a town or city is adjacent to a freshwater lake or major river, most urban water supplies require conveyance of water from water sources some distance away as the original siting of the urban area was not determined on the basis of such things as water demands.

Exceptions to this can be where there are relatively large quantities of economically exploitable groundwater in the vicinity of the town, although even here and as the town develops, so does the risk of pollution.

Generally speaking however, the supply of water to urban areas has been a progressive development over a number of years and today's Designer is usually faced with the need to augment and / or rehabilitate or replace parts of an aging infrastructure.

In so doing, it is important to consider the existing infrastructure and to be guided by past success and failures. This needs to consider all system components from intakes to treatment to pumping plant to storage and to pipework.

If a past component has been singularly successful in attaining its design life or in providing a low maintenance solution its appropriateness should be recognised. Conversely if a scheme component has failed in either of these regards, then this needs careful consideration before repeating the mistake. Sometimes it was over-sophistication, sometimes it was use of an inappropriate material or inadequate attention to installation, sometimes it was failing to take account of likely changing circumstances brought about by the installation itself.

Designers should investigate all such occurrences professionally and then recommend on the way forward, possibly recommending straight forward duplication in the case of previous success, or exclusion from consideration in the case of past failure, or a change in design approach or manufacturing specification or installation procedure where this was deemed to have been at fault previously.

4.5 WATER BALANCE

To understand all the various components that make up Gross Water Supply or System Input Balance, it is necessary to look at the complete Water Balance. Because of the increasing need to take serious account of the amount of water that cannot be charged for all such water volumes

[< >] See for example the Hai Water Project in Kilimanjaro Region.
Refer also to the manual on Management Models for Rural Water Supply Schemes under WSDP.

are best referred to as Non Revenue Water (NRW). The use of the earlier term of UfW or Unaccounted for Water is being used less and less because of varying definitions worldwide.

It is therefore recommended that the Water Balance as recommended by the International Water Association (IWA) be adopted here as indicated in the following table.

TABLE 4.2: THE IWA 'BEST PRACTICE' STANDARD WATER BALANCE

<table>
<tr><td rowspan="9">System Input Volume (corrected for known errors)</td><td rowspan="4">Authorised consumption</td><td rowspan="2">Billed Authorised Consumption</td><td>Billed Metered Consumption (including any water exported)</td><td rowspan="2">Revenue Water</td></tr>
<tr><td>Billed Unmetered Consumption</td></tr>
<tr><td rowspan="2">Unbilled Authorised Consumption</td><td>Unbilled Metered Consumption</td><td rowspan="7">Non-Revenue Water (NRW)</td></tr>
<tr><td>Unbilled Unmetered Consumption</td></tr>
<tr><td rowspan="5">Water losses</td><td rowspan="2">Apparent Losses</td><td>Unauthorised Consumption</td></tr>
<tr><td>Customer Metering Inaccuracies</td></tr>
<tr><td rowspan="3">Real Losses</td><td>Leakage on Transmission and/or Distribution Mains</td></tr>
<tr><td>Leakage and Overflows at Utility's Storage Tanks</td></tr>
<tr><td>Leakage on Service Connections up to point of Customer Metering</td></tr>
</table>

4.6 DETERMINING AUTHORISED CONSUMPTION

4.6.1 Water Demand Projections

One of the most difficult tasks facing a Designer is that of determining Water Demand. Another is quantifying Water Losses that must be allowed for. Under estimation will result in failure to achieve the design life of a scheme whilst over estimation, especially of water demand, will tie up scarce financial resources unnecessarily.

In a sector that is rapidly moving away from subsidised flat rate tariffs, unmetered supplies, and lax attention to defaulting customers towards application of commercial tariffs, consumer metering, and rigorous disconnection policies for non-payment, the use of historic data for water demand will result in over-estimation. On the other hand, water losses have often been grossly underestimated.

Because of the lack of appropriate historic data on water demand, too rigorous a move in the opposite direction will result in under-estimation and the need for further investment somewhat sooner than planned, unless water losses are also carefully considered and allowed for.

At present therefore, a range of per capita demand figures is put forward here for consideration. They are a guide but should not be followed blindly where more suitable information is at hand.

4.6.2 Metering and Demand Management

The inclusion of bulk or zonal meters for control, and for leakage and theft management and consumer meters for metering of customers as well as the use of tariff structures to facilitate demand-management should now be an integral part of any Design consideration.

Appropriate meter selection, especially of consumer meters is of considerable importance in this, otherwise their retrospective introduction into existing systems is almost certainly doomed to failure. No mechanical meter likes dirty water, very low pressures and intermittent supply where water and then air has to be passed through the meter. All mechanical meters are prone to vandalism but some less so than others. Wet dial meters are less prone to vandalism than dry dial meters but more likely to fail in a poor supply situation.

On existing piped water supply schemes and in rural areas in particular where consumers have got used to an unmetered supply, those fortunate enough to be at the upper ends of such schemes will have got used to drawing water for a variety of agricultural and livestock watering purposes never allowed for by the original designers and will be extremely reluctant to reduce their water requirements when metered. Meter tampering and vandalism is then all too often the result and this then requires the inclusion of a project software or facilitation component to educate consumers, increase the awareness of their actions and reduce such risks. It further requires approaching the problem from the grass-roots level and placing the community at the forefront of the complete planning, design and subsequent ownership and operation process.

To technically assist in this, there are on the market today electronic consumer meters that have no moving parts and do not record air [< >]. They are however still considerably more expensive than all types of mechanical meters. However and especially where the number of consumers is limited, there use should be seriously considered.

Whilst metering should be considered as an essential part of future water supplies, there remains the question of source water availability, investment cost and management of demand. Even where the Client does not specifically require the issue of demand management to be taken into account; Designers should consider several different scenarios to allow decision makers within the Clients organisation to make an informed decision on this.

4.6.3 Domestic Consumption

The amount of water consumed depends in part on the level of service provided. It is at its lowest when water is distributed through domestic points (public taps or kiosks) some walking distance from the house. When the water is brought to the house by piping· the consumption increases considerably. It is necessary to note also that there is no incentive for the consumer to save water when water is supplied at a flat monthly rate . With waterborne sanitation and high standard of inside installations (bath, washing machine etc) the per capita consumption may be ten times more than from a public tap.

The proposed figures for water consumption rates as given in the table 4.3 are a guide for design but may be adjusted as necessary after considering the particular conditions and consumption trend of the area.

[< >] For example the SmartMeter™ family of fluidic oscillation water meters which meets AWWA C713-05 (Design and Quality Standard for Cold-water Fluidic Oscillation Meters). Such solid-state water meters provide accurate measurement of water flows throughout the entire product lifetime which can be as long as 20 years. They are not affected by wear and subsequent accuracy problems that are caused by grit and other particulates, whilst an air detection system prevents the measurement of air-flow.

To make an estimate of the future water requirements, it is necessary not only to decide upon the per capita consumption figures to be used but also the percentage of population served from public taps and of those served by different categories of house connections. It is also necessary to consider saturation-density of a given area because once all plots are built upon, the growth rate in the population in the area will slow dramatically.

TABLE 4.3: WATER REQUIREMENTS

CONSUMER CATEGORY	RURAL AREAS (l/ca/d)			URBAN AREAS (l/ca/d)			REMARKS
	FR	M-UT	M-PBT	FR	M-UT	M-PBT	
Low income using kiosks or public taps	25	25	25	25	25	25	Most squatter areas, to be taken as the minimum
Low income multiple household with Yard Tap	50	450	40	50	45	40	Low income group housing No inside installation and pit latrine.
Low income, single household with Yard Tap	70	60	50	70	60	50	Low income group housing No inside installation and pit latrine
Medium Income Household				130	110	90	Medium income group housing, with sewer or septic tank.
High Income Household				250	200	150	High income group housing, with sewer or septic tank.
FR = flat rate; M-UT = metered with uniform tariff; M-PBT = metered with progressive block tariff.							

Even if the target is to serve all the people in the design area with water, there may remain a certain number of people that have to rely on their own sources as they may be too far away to be served from the organized water supply. The percentage of those people tends to decrease with time when services of public water supply become closer due to extensions and as the financial constraints decrease and the supply becomes more reliable.

The present situation can be determined by site visits, studying the records of the present water supply organization, and from aerial photography and archived satellite imagery, etc. At present there are two levels of such imagery that need to be considered. The first is country- wide and either LandSat-7, or GeoCover 14.25 metre geo-referenced ortho-rectified imagery can be acquired as a basis for assessment and mapping. This has been widely archived since 2000 and is now very modestly priced at about US$ 0.5 per km^2.

For detailed planning and design and the actual counting of buildings in urban areas, either the aerial photographs taken in preparation for the 2002 Population and Household Census or QuickBird 60cm satellite imagery can be used. Both enable maps to be produced at scales of 1:2000 and better and archived QuickBird imagery is increasingly available, archiving commencing in 2002. For major projects and where archived imagery is not yet available, consideration can also be given to ordering new image acquisition, however this is expensive in comparison.

The extent of availability of Landsat imagery can be checked by those with Internet access from such organisations and Digital Globe / Map Mart and that of QuickBird archived imagery can also be found by using ‘Google Earth’. Before ordering archived satellite images however it is important to check on % cloud cover and for the QuickBird imagery on environmental quality also.

For future forecasting it is necessary to find out the proposed kind of dwellings and the standard of services zone by zone. Here contacts with the local Planning Officers will be useful with regards to such things as urban master plans.

In rural areas, the primary target is to provide the entire population with access to an improved water source, (65% by 2010 and 95% by 2025) whilst the secondary target must be to reach as many persons as possible with a controlled water supply.

Where possible, consideration should be given to use of shallow wells or protected springs and then boreholes. However recent climatic patterns suggest that shallow wells may only be an interim solution and where possible deeper groundwater may be a better first option. Progressive development of groundwater sources should also be considered, starting with an appropriate hand pump lifting the water to a distribution point at the surface, or to an adjacent elevated tank. Subsequently mechanising the pump, possibly using solar power and/or a wind generator to run a small motor, and then piping the water from the tank to additional distribution points can be considered where sufficient maintenance skills exist.

With the increasing involvement of NGOs and small private entrepreneurial operators such abilities will develop over time.

As a general rule and in order to make the most efficient use of the limited financial and technical resources available, the first phase of any piped rural water supply should principally be designed for domestic points and for the same reason wherever possible shallow wells should be used as the source. Customer connections should only be provided-for-schools, dispensaries and offices and only in exceptional cases for private houses. Water for livestock should be considered only where feasible and where livestock keeping is a principle livelihood or income source for the local population.

The provision of a water point at schools wherever possible is considered extremely important, not only to facilitate hand washing as part of hygienic sanitation but also to allow the girl child in particular to carry water straight from school to home after studies. In this regard, Designers are referred to the World Bank / UNICEF toolkit [< >].

In urban areas the final target is to provide house connections for every dwelling. Once again, however, the practical way is advancing by phases so that for the first phase the water will be supplied mainly by domestic points.

As an example Table 4.4 shows typical percentages (%) of different categories of water consumers in different consumer areas. It should only be used where more specific information is unavailable.

< > Web-based Toolkit on Hygiene, Sanitation and Water in Schools published by the World Bank, 2005

TABLE 4.4: CATEGORIES OF URBAN WATER CONSUMERS

CLASSIFICATION as per Section 4.3.3	INITIAL (CURRENT) DP	LC	MC	HC	FUTURE (10 YEARS) DP	LC	MC	HC	ULTIMATE (20 YEARS) DP	LC	MC	HC
Rural Service Centre	60	20	15	5	50	25	20	5	40	30	20	10
District Centre	50	25	15	10	40	30	20	10	30	35	25	10
Municipal Centre	40	30	20	10	30	35	25	10	20	35	30	15
City Centre	30	35	25	10	20	40	30	15	10	40	35	15
DP = Domestic Point; LC = Low Cost Housing; MD = Medium Cost Housing; HC = High Cost Housing												

4.6.4 Agricultural and Livestock Requirements

Water requirement for livestock will be included in rural water supply designs where feasible, however, emphasis should be placed on the use of dams, charcos and water wells for livestock.

4.6.4.1 Definitions

For the sake of water demand for livestock the following grading of domestic animals in terms of a stock unit is to be used where one stock unit is equivalent to:

one head of cattle,
or two donkeys,
or five goats or five sheep (sometimes referred to collectively as shoats),
or thirty head of poultry (hens, ducks, geese).

Special cases are high grade dairy cows where one cow is equal to 2 (or 3), stock units.

Present livestock numbers can be found either by counting or inquiry and then converted into stock units.

Future populations of livestock may be taken at 25% growth in 10 years and 50% growth in 20 years provided the carrying capacity of the land allows it or otherwise the present figures can be taken for the future also. This decision should be reached in consultation with the competent authorities. The water demand for livestock can be calculated using 25 l/stock unit per day.

The annual growth rate for livestock may further be taken as 2.6% and 2.0%· for sheep and goats respectively. When deciding upon whether or not to cater for the water requirements for livestock, serious consideration should be given to the alternatives of natural water in ponds, streams, waterholes, etc. in the neighbourhood for the use of stock watering, so that the water demands on the water supply system can be eased and the overall flow and consequently the cost can be reduced.

Agricultural use of piped water supplies is generally not encouraged as the demands will be out of all proportion to the other combined demands. However and in exceptional circumstances it may be considered for high value horticultural crops but only when the 24 hours flow is sufficient to meet all other demands; then irrigation demand will be additional. In such circumstances, private storage must be arranged.

What is of prime importance when considering both livestock watering and horticultural irrigation is that the costs of construction and O&M of such a scheme should still be within the affordable limits of the community.

4.6.4.2 Livestock Carrying Capacities

Determination of livestock carrying capacities is in part a seasonal matter and is likely to change over time. It needs to be checked in the project area with the appropriate livestock authorities. However, the figures given in the following table may be used as an initial guideline [< >].

For more detailed consideration, the subsequent discussion on various influencing factors should be taken into account.

TABLE 4.5: GRAZING LAND BY RAINFALL ZONE

Zone	Average Rainfall in mm/year	Approx. land area in sq. km	Potential Stocking: ha per stock unit	Potential Livestock Numbers:
Arid		**195,000**	**More than 4**	**4,300,000**
Semi-arid	**300- 750**	**175,000**	**2 -4**	**5,800,000**
Dry-sub humid	**750-1,000**	**165,000**	**1 -3**	**8,300,000**
Humid	**> 1000**	**50,000**	**0.5-2**	**1,600,000**
Afro-alpine		**15,000**	**-**	**-**
Total		**600,000**	**-**	**20,000,000**

General climatic effects of topography

Climate in Tanzania is diverse as a result of: a) The proximity of the ocean and inland lakes; b) Altitude, which governs temperature; and c) Latitude. The diversity of topography and other factors give rise to a range of average rainfall from 200 – 2,000 mm per annum. Most of the country receives less than 1 000 mm, except highlands and parts of the extreme south and west where 1,400 – 2,000 mm can be expected. In the central arid areas 200 - 600 mm falls on average.

Rainfall tends to be unimodal throughout most of the country except in the extreme north and east where the pattern is more bimodal as indicated on the map opposite: -

Table 4.6 on the following page is a guide to the rainfall seasons in the north, central and southern zones.

< > Country Pasture / Forage Resource Profiles – United Republic of Tanzania by Dr. Sebastian Sarwatt and Dr. Esther Mollel, Dept. of Animal Science, Sokoine University of Agriculture, 2000. (livestock data modified by S.G. Reynolds in August 2006)

TABLE 4.6: RAINFALL SEASONS IN TANZANIA

Month	Wind direction	Season	Zone		
			North	Central	South
December to March	NE	Kaskazi	Dry	Dry	Wet
March to May	Variable	Masika	Wet	Wet	Wet
June to September	SE	Kusi	Dry	Dry	Dry
October to November	NE	Vuli	Wet	Wet	Wet

Two additional factors modify the effectiveness of rainfall, its reliability is low, even in areas of high average rainfall and very high evapotranspiration reduces its effectiveness, especially in the semi-arid areas. Drought is a major factor in livestock production especially in Arusha, Dodoma, Singida, Shinyanga, Tabora and Mwanza.

Agro-ecological zones and major agricultural enterprises:

The combined influences of altitude, latitude, rainfall and soil determine the climatic zones of East Africa, and classify Tanzania into five zones. The great altitude range of agriculture means a great diversity of cropping systems from the coconut groves of the coast up to cool-area crops such as pyrethrum and wheat.

a) Afro-alpine: 1 percent of area: Afro-alpine moorland and grassland, or barren land, above the forest line; of limited use and potential, except as water catchment and for tourism.

b) Humid to dry sub-humid: 9 percent of area: Forest-derived grasslands and bush with potential for forestry or intensive agriculture, including pyrethrum, coffee and tea. Natural grassland responds to intensive management and can support one stock unit on less than a hectare.

c) Dry sub-humid to semi-arid: 30 percent of area: not of forest potential with a variable cover of moist woodland, bush or savannah; trees mostly Brachystegia or Combretum. Agricultural potential is high, large areas are under extensive grazing; stock carrying capacity can be high, under two ha per stock unit. Regular burning may be necessary.

d) Semi-arid: 30 percent of area: Land of marginal crop potential limited to sisal or quick maturing cereals, carrying natural vegetation of Acacia-Themeda association but including dry Brachystegia woodland. This is potentially productive grazing at less than 4 ha per stock unit, limited by bush encroachment, leached soils, inadequate water and tsetse fly infestation

e) Arid: 30 percent of area: Unsuitable for agriculture, except with fertile soils and run- on rainfall. Typically on pasture dominated by Commiphora, Acacia and perennial grasses such as Cenchrus ciliaris and Chloris spp. over 4 ha is required per stock unit and wild life is important. Burning requires care but can be highly effective in bush control.

Grazing lands

Natural pasture provides over 90 percent of the feed requirements of ruminant livestock in Tanzania. It is very diverse owing to the wide variety of ecological variations throughout the country. Five main pasture types have been identified by ecological zone.

TABLE 4.7: SUMMARY OF GRAZING RESOURCES IN TANZANIA.

Land use 000 ha	Semi arid to sub humid	Humid plateaux	Humid low land	Very humid high land	Very humid low land	Total
Total area	**22,560**	**32,208**	**20,062**	**12,235**	**1,324**	**87,389**
Cultivated	**1,026**	**1,697**	**1,160**	**472**	**168**	**4,523**
Grazing	**15,684**	**15,022**	**8,464**	**3,813**	**896**	**43,879**

a) Semi arid to sub humid grazing land: covers nearly 30 percent of the grazing area and is mainly found in the central plains including the pastoral systems of Arusha, Dodoma, Shinyanga and Singida. About 40 percent of the national cattle herd are found here at density of less than three ha/head. Seasonality of production, drought and overgrazing are major problems. The trees mostly are *Brachystegia* or *Combretum spp*. The commonest grasses include *Chloris gayana*, *Cenchrus ciliaris*, *Brachiaria brizantha*, *Cynodon spp.* and *Andropogon gayanus*. *Sporobolus spp.* dominate in overgrazed areas.

b) Humid plateau lands: these represent another 30 percent of the grazing area and support nearly 50 percent of the cattle. They are typified by the agropastoral zones of Mwanza, Mara and Mbeya. These two types, represent 60 percent of the area and carry 90 percent of the stock. The most common species of legumes found in this zone are *Desmodium spp.*, *Clitoria ternatea*, *Macroptilium atropurpureum.*, *Neonotonia wightii* and *Stylosanthes guianensis*. Dominant grasses are *Chloris gayana*, *Pennisetum purpureum* and *Setaria sphacelata*.

c) Humid lowland: represents 20 percent of the grazing, but only about two percent of the livestock are here. The regions with the most potential are Mtwara and Lindi. Species commonly found include *Hyparrhenia spp.* and *Cynodon spp*.

d) Very humid highlands: cover nine percent of the area and support five percent of the cattle. They are in parts of Kilimanjaro, Mbeya, Ruvuma and Kagera; most of the exotic and crossbred cattle are here. The potential of the area is for forestry or intensive agriculture including pyrethrum, coffee and tea. The natural grassland responds to intensive management and can support one stock unit on less than one hectare. Grasses found in the zone are *Cenchrus ciliaris*, *Setaria sphacelata var. splendida, Panicum spp.*, and *Pennisetum purpureum*, and legumes *Centrosema pubescens*, *Desmodium intortum*, *Neonotonia wightii* and *Medicago sativa.*.

e) Very humid lowlands: This is a limited area, restricted to Tanga region, but, livestock, especially crossbred dairy stock is increasing. Grass species commonly found in include *Panicum spp.*, *Pennisetum purpureum*, and *Chloris gayana*. *Neonotonia wightii* and *Centrosema pubescens* are the major legumes.

The recommended consumption amounts for livestock are as follows:

TABLE 4.8: CONSUMPTION RATES FOR LIVESTOCK

CONSUMER	UNIT	CONSUMPTION RATE in l/d	REMARKS
Livestock	1/animal/d	50 – 90	High grade dairy cattle
		25	Local breed cattle
		5	Sheep and Goats
		12.5	Donkeys

4.6.5 Other Water Requirements

4.6.5.1 General

Whereas the water requirements of rural areas are more or less confined to population, institutions such as schools, market centres and livestock and to some extent health facilities, the water requirements for urban areas are governed by various factors.

In addition to population requirements, an urban water supply has to cater, for the needs of public and private institutions, industries, commercial activities, fire fighting aspects etc.

4.6.5.2 Institutional Water Demands

Public and private institutions include: Schools, Hospitals, Administration Offices, Police, Missions, Churches and Mosques, Prisons, etc. In Table 4.9, some figures for institutional water demand are given. The water requirements for staff working in the institutions should be estimated separately in the same way as for other domestic water consumption.

If large demand units are included in the scheme, such as Universities, major hospitals, boarding schools etc., a special study of their water requirements is recommended instead of using the average figures given in the table below.

TABLE 4.9: INSTITUTIONAL WATER DEMANDS

CONSUMER	UNIT	RURAL 1/d	URBAN 1/d	REMARK
Schools – Day Schools	1/std/d	10	10 25	With pit latrine With WC
– Boarding Schools	l/std/d	70	70	With WC
Health care Dispensaries	1/visitor/d	10	10	Out patients only
Health	1/bed/d	50	50	No modern facilities
Health	l/bed/d	100	100	With WC and sewer
Hospitals, District	l/bed/d	-	200	With WC and sewer
Hospitals, Regional	1/bed/d	-	400	With surgery unit
Administrative Offices	1/worker	10	- 70	With pit latrines With WC

4.6.5.3 Industrial Water Demands

The water· consumption in industry varies considerably depending on the kind and size of the- industry. There are dry industries which consume virtually no water in their processes, and the only water consumption is that for staff and cleaning of the premises. On the other hand the water requirements for wet industries such as for a paper or cotton processing factory can be a great deal. The following table gives some examples of the water consumption in different kind of industry.

For existing industry, the water consumption can be found out by checking their metered consumption or if there are-no records available by estimating according to the kind and size of production. The consumption figures for larger units must always be based on proper measurements not on estimates.

For future small industrial demands allowance can be made using the following table:

TABLE 4.10: SPECIFIC INDUSTRIAL WATER REQUIREMENTS

INDUSTRY	PRODUCT OR RAW MATERIAL UNIT	WATER CON-SUMPTION IN m^3 PER UNIT OF RAW MATERIAL
Food Industry		
Dairy	Milk received (1000 1)	2 – 5
Abattoir	Animal slaughtered (gross weight)	4 – 10
Brewery	Beer (1000 1)	10 – 20
Sugar	Cane (tonne)	10 – 20
Wood processing industry		
Pulp mill	Bleached pulp (tonne)	100 – 800
Paper mill	High quality paper (tonne)	300 – 450
Chipboard factory	Chipboard (tonne)	50 – 150
Others		
Tannery	Raw skins (tonne)	50 – 120
Cotton mill	Cotton thread tufi	50 – 150

Larger and future development of industrial water requirements have to be found out by direct interviews of the technical management of existing industries and by contacts with the local planning officers and local government officials, e.g. municipal council officers. For future industries to be established, Ministry of Industries and Trade, the Regional Planning Officer or organizations such as the National Development Corporation (NDC), Small Industries Development Organization (SIDO) and owners of private industries shall be consulted.

Where there is only a reservation for an industrial area in the town plan but without any specifications; estimates of the future water requirements can be based on the figures below:-

TABLE 4.11: INDUSTRIAL WATER DEMAND (m3 /ha/d) FOR FUTURE INDUSTRIES

INDUSTRY TYPE	WATER DEMAND m^3 /ha/d
Medium Scale (water intensive)	50
Medium scale (medium water intensive)	20
Small scale (dry)	5

A basic or semi-domestic type of consumption of 25 l/ca/d is allowed for in the above figures.

If the requirement of a particular industry is large, then separate local sources must be examined whilst a part of the requirement can be supplemented from the town water supply system. The fire fighting water requirements for industrial areas must be estimated separately.

The principle should be that if the fire fighting water demand is bigger than what is normal capacity of the distribution system, the industry in question must provide its own water reserve for fire fighting.

4.6.5.4 Commercial Water Demands

Commercial water consumption occurs in hotels, restaurants, bars, shops, small workshops, service stations, etc. Their present water demand should be known by their metered water consumption, but at least the bigger hotels, restaurants and services stations must be checked. Future water requirements can be based on the estimated development of this sector. Table 4.12 gives water consumption figures for hotels and restaurants.

If there is only a reservation in the town plan for the future business area without any specification, the estimate must be based on per hectare demand. As a guide, a water demand of 10 – 15m^3/ha/d for a non specified commercial area in a new town plan can be adopted.

TABLE 4.12: COMMERCIAL WATER REQUIREMENTS

CONSUMER	UNIT	RURAL 1/pd	URBAN 1/pd	REMARKS
Hotels	1/bed/d	70	70 200 400	Low class Medium class High class
Bars		70	70 100 300	Low class Medium class High class
Shops		25	70 130	

4.6.5.5 Net Water Demand (revenue water)

By summating all the water demands as discussed in sections 4.5.4 and 4.5.5 and dividing by the design population the net water demand per capita can be determined. This is then the potential billable water consumption or revenue water.

4.6.5.6 Water for Fire Fighting in Urban Areas

Fire fighting requirements are only necessary in Urban areas. The water supplied here normally forms part of the unbilled authorised consumption.

(i) Minimum pipe diameters, flows, pressures and hydrant distribution

Pipe sizes in the non-industrial fire fighting distribution system should not be less than DN80 but where there are parallel distribution mains down both sides of a road, the distribution main without hydrants may be DN65. In mixed or general industrial areas the minimum diameter should be DN110/100, whilst on industrial estates it should be DN150.

The design flow into a fire hydrant should not be less than 10 l/s. The residual head in the pipes at a hydrant should not be less than 15m. The distance between two adjacent fire hydrants should not exceed 300m such that the distance of a building from a hydrant is not more than 150m.

(ii) Fire fighting reserve should be in accordance with the number of people served by a reservoir as shown in the following table:

TABLE 4.13: FIRE FIGHTING RESERVE

POPULATION SERVED BY THE RESERVOIR	POSSIBLE NUMBER OF SIMULTANEOUS FIRES
Less than 10,000	**1**
10,000 to 50,000	**2**
Above 50,000	**3**

(iii) Dimensioning flows and reservoir storage are as follows: -

TABLE 4.14: FIRE FIGHTING REQUIREMENTS

CATEGORY	FIRE FIGHTING REQUIREMENTS		WATER STORAGE RESERVOIR IN m^3
	FLOW IN 1/s	PERIOD OF FLOW IN hrs	
" A "	10	2	100
" B "	10 - 30	4	200 – 400
" C "	Water demand to be considered individually according to the activities, value of property, sensitivity to fire etc. of the area.		
"A" = residential areas; "B" = multi storey houses, office and commercial areas; "C" = industrial areas and other important but highly inflammable areas such as airport, hospitals, oil depots etc.			

The determination of fire fighting requirements is an extremely complex issue so that a simplified approach is proposed as follows[< >]:

TABLE 4.15: FIRE FIGHTING FLOWS

CATEGORY	DESCRIPTION		MINIMUM FLOW IN l/s
housing	Housing developments with units of detached or semidetached houses of not more than two floors should have a water supply capable of delivering a minimum through any single hydrant of:		8
	Multi occupied housing developments with units of more than two floors should have a water supply capable of delivering a minimum through any single hydrant on the development of:		20 - 35
transportat-ion	Lorry/coach parks, multi-storey car parks and service stations	All of these amenities should have a water supply capable of delivering a minimum through any single hydrant on the development or within a vehicular distance of 90 metres from the complex of:	25
industry	In order that an adequate supply of water is available for use by the Fire Authority in case of fire it is recommended that the water supply infrastructure to any industrial estate is as follows with the mains network on site being normally not less than DN150	Up to one hectare	20
		One to two hectares	35
		Two to three hectares	50
		Over three hectares	75

[< >] National guidance document on the provision of water for fire fighting, Appendix 5, Water UK, May 2002

CATEGORY	DESCRIPTION		MINIMUM FLOW IN l/s
shopping, offices, recreation and tourism	Commercial developments of this type should have a water supply capable of delivering a minimum flow to the development site of between:		20 - 75
education, health and community facilities	Village and small community halls	Should have a water supply capable of delivering a minimum flow through any single hydrant on the development or within a vehicular distance of 100 metres from the complex.	15
	Primary schools and single storey health centres	Should have a water supply capable of delivering a minimum flow of through any single hydrant on the development or within a vehicular distance of 70 metres from the complex.	20
	Secondary schools, colleges, large health and community facilities	Should have a water supply capable of delivering a minimum flow through any single hydrant on the development or within a vehicular distance of 70 metres from the complex.	35

Alternatively, and where more precise figures are required, the following empirical formula may be used to obtain the fire flow requirements.

<u>Formula used by the Insurance Service Office, USA</u> [< >]. The formula is highly structured relying heavily upon the input of premise and occupancy data that needs to be collected and is therefore extremely time consuming to apply. Its use results in flows that can be as much as three times higher than those used in the table above. It is however considered appropriate where large diameter fire hoses and relatively low pressures occur. Its use should however be limited to Central Business Districts, high value property areas and areas where a considerable amount of flammable material such as timber is used in building construction. In its metric version it is:

$$F = 3.7C\,(A)^{0.5}$$

Where, F = required fire flow in litres per second; C = coefficient related to type of construction: 1.5 for wood frame buildings, 1.0 for ordinary construction, 0.8 for non combustible material and 0.6 for fire resistance construction; A = total floor area in square metres including. all stories in the building, but excluding basements.

For fire resistive buildings, the six largest successive floor areas are used if the vertical openings are unprotected; but where the vertical openings are properly protected, only the three largest successive floor areas are included.

Regardless of the calculated value; the fire flow shall not exceed 500 l/s for wood frame, or ordinary construction, or 380 l/s for non combustible or fire resistive buildings; except that for a normal one story building of any type it may not exceed 380 l/s. The fire flow shall not be less than 30 l/s. For groupings of single family and small two family dwellings not exceeding two storeys in height the fire flows in the Table 4.16 below:.

[< >] Guide for Determination of Needed Fire Flow, Insurance Service Office, 2006, (www.iso.com) 545 Washington Boulevard, Jersey City, New Jersey, USA, ISO (4476), Edition 05-2006

TABLE 4.16: RESIDENTIAL FIRE FLOWS

DISTANCE BETWEEN ADJACENT UNITS (m)	REQUIRED FIRE FLOW (l/min)
More than 30.5	1,890
9.5 – 30.5	2,835 – 3,780
3.4 – 9.2	3,780 – 5,670
Less than 3.0	5,670 – 7,560

The duration of fire flow that should be sustained is as follows:

TABLE 4.17: FIRE FLOW DURATION

REQUIRED FIRE FLOW IN l/s	DURATION IN MINUTES
Less than 3,780	4
3,780 – 4,725	5
4,725 – 5,670	6
5,670 – 6,615	7
6,615 – 7,560	8
7,560 – 8,505	9
More – 8,505	10

However since such a high rate is exerted for a short while by which time the fire either dies down or is brought under control and since the instance of fires are relatively few in number each year, the per capita water required is either calculated on yearly basis or may be ignored.

The required number of fire hydrants should be provided at suitable locations in the distribution systems so that the specified number of fire hydrants can be used during the emergency according to the requirements

4.6.5.7 Operational Demands

Operational demands include the water required for fire fighting, if this is determined, that required for operation of water treatment processes such as clarifier de-sludging, filter backwashing and chemical mixing, and operational activities such as the flushing out of reservoirs and the pipework system through washouts and when cleaning bulk meter screens.

Excluding fire fighting the suggested percentages are 5% for water treatment where it takes place and a further 2% for other operational demands.

4.7 SYSTEM LOSSES

In any design it is necessary to allow for water losses that are likely to occur. These will tend to increase over time and depend on a number of conditions.

Technical water losses occur due to leakages and overflow from reservoirs, treatment units, valves, mains and distributions piping. Traditionally it was suggested that overall, this can be taken at between 20 to 25% of the gross water demand (gross supply). However, experience shows that in urban areas except under the best situations, this can grossly underestimate what actually occurs.

Other losses result from third party damage, usually resulting from either successful or unsuccessful attempts at illegally obtaining water for consumption.

In the case of pipework, loss will relate to inadequacy of design, poor pipe selection, poor quality of manufacture and installation, operating pressure and in urban areas in particular, risk of third party damage including vandalism. In a zero-failures cost approach the loss due to these elements can be equated to zero.

Losses of water due to negligence of water consumers, unauthorized abstractions from the network, third-party damage including vandalism etc, have in the past rarely been considered for design purposes. However to ignore them, passing them off as an operation and maintenance problem that should be controlled by those in authority is but to pretend the problem does not exist. Not only in estimating revenue should this element be considered but in designing, specifying and implementing a project. Again and where the zero-failures cost approach has been adopted the losses attributable to vandalism should be minimal and can also be equated to zero.

Vandalism and illegal connections vary enormously. In small Tanzanian towns with a continuous water supply, relatively small proportion of urban poor and good control it can be quite small. In larger towns, with large populations of urban poor and irregular or rationed supply it can be very significant.

For new pipes in urban distribution networks and in the absence of other data to the contrary, and assuming a working pressure of 5 bar, and the use of appropriate saddle clamps for consumer connections, the following is suggested: -

TABLE 4.18: TYPICAL URBAN DISTRIBUTION LOSSES FOR DIFFERENT PIPES

Pipe	Minimum Cover (mm)	Loss from Pipes & Joints	Third Party Damage to Pipeline	Loss at Consumer Connection	Suggested Total Design Loss in Pipe Material
D.I.	900 / 600	8%	2%	2%	12%
PE $_{weld}$	900	0%	10%	0%	10%
PE $_{weld}$	1500	0%	5%	0%	5%
Steel	900 / 600	8%	5%	2% [15%]	15%
PE $_{comp}$	900	5%	12%	3% [15%]	18%
PE $_{comp}$	1500	5%	6%	3% [15%]	14%
PVCu	900	8%	15%	3% [15%]	26%
PVCu	1500	8%	5%	3% [15%]	16%
Cover to D.I. and Steel, 900mm in primary mains and 600mm in secondary mains					

If ferrules alone or rigid saddle clamps are to be used for consumer connections subsequent to laying in steel, PE, or PVCu, design loss at such connections should be as shown in [ ] brackets.

TABLE 4.19: URBAN DISTRIBUTION LOSSES IN CONSUMER CONNECTIONS, PER 50 M LENGTH (OR PART THEREOF)

Pipe	Minimum Cover (mm)	Primarily due to Vandalism Ave (range)	Primarily due to Illegal Tapping Ave (range)	Suggested Design Loss/50 m length in Pipe Material
PE $_{weld}$	900	3% (0% to 6%)	2% (5% to 4%)	5%
PE $_{weld}$	1500	2% (0% to 4%)	1% (0% to 2%)	3%
Steel	600	1% (0% to 2%)	2% (1% to 3%)	3%
PE $_{comp}$	900	3% (1% to 5%)	4% (2% to 6%)	7%
PE $_{comp}$	1500	2% (1% to 3%)	1% (0% to 2%)	3%

4.8 GROSS WATER DEMAND AND REVENUE WATER

Water demand may be defined as the gross average Daily Demand = Net total demand / 'NRW' This is the Gross Supply, where Net total demand = no of consumers x per capita consumption.

Non Revenue Water (NRW) is as described in the Water Balance in section 4.5.

Based on the forgoing, it is possible to draw up a percentage design water balance stream for a scheme, with the various elements presented in tabular form. An example is presented below for a D.I. – PVCu – PEHD scenario without treated water transmission mains and with consumer connections n.e. 50 m in length.

TABLE 4.20: TYPICAL DESIGN WATER BALANCE STREAM SHOWING REPRESENTATIVE LOSS RANGE AND VALUES:

ITEM	DESCRIPTION	WATER BALANCE ITEM	LOSS RANGE %'s	SELECTED FOR DESIGN %
(i)	System Input Volume (Production)		100%	100%
(ii)	Intake and Raw Water Transmission	RL	1% - 3%	2%
(iii)	Treatment Plant losses	UAC	4% - 6%	5%
(iv)	Treated Water Transmission Leakage	RL	n/a	n/a
(v)	Transmission Third Party Damage	RL	n/a	n/a
(vi)	Balancing Storage Losses	RL	0%-2%	1%
(vii)	**Treated water to Supply**		**95% -89%**	**92%**
(viii)	Operational requirements in the system	UAC	1% - 3%	2%
(ix)	Primary distribution system losses	RL	12%	12%
(ix)	S'ndr'y distribution system losses, & at Cc	RL	16% -26%	16%
(x)	Leakage in Consumer connection (Cc)	RL	2% - 7%	5%
(xi)	Customer Metering Inaccuracies	RL	1% - 3%	2%
(xii)	Real Losses, (variable depending on the materials selected and efficiency of the system)	∑RL	37% - 62%	45%
(xiii)	**Billable Water (Revenue Water)**	**RW**	**63%-38%**	**55%**
RL = real loss; UAC = unbilled authorised consumption; RW = revenue water				

The results may be considered as alarming but illustrate why there is need to make a very serious evaluation at the design stage as to pipe materials and installation.

Then system efficiency = treated water – billable water / treated water, and
total water demand: = domestic + livestock + institutional + other uses + NRW

4.9 VARIATIONS IN WATER CONSUMPTION

4.9.1 Definitions

Whilst water demand is normally calculated according to the average requirements, actual consumption varies from hour to hour and from day to day. Due to this non uniformity of water demand, provision needs to be made in different units of the water supply system to cater for these variations.

In order to evaluate the importance of variation in water consumption the following definitions are relevant:

(i) Average Daily Demand

Q_{da} = The result of adding together average domestic, livestock, institutional commercial, and industrial water average requirements.

(ii) Maximum Daily Demand

Q_{dmax} = The result of the multiplication of the average daily demand by the peak day factor – K_d. It represents the demand on the day in the year in which the maximum consumption is recorded.

(iii) Peak Hour Demand

Q_{hmax} = The result of multiplying the maximum daily demand by the peak day factor – K_h. It represents the peak hour flow during the day with maximum consumption.

(iv) Guiding Figures for Peak Factors

$K_d = Q_{dmax} / Q_{da}$ = peak day factor

$K_h = Q_{hmax} / Q_{dmax}$ = peak hour factor

For design purposes, the peak factor shall be selected by giving consideration to the size and kind of the scheme and services required.

Generally an intake and the main from the intake to a treatment works is dimensioned to meet the **peak day ultimate demand**. For gravity schemes this means a main designed for a flow during 24 hours, while for pumping main the design flow is according to the pumping hours decided upon.

This gives the minimum size of pipe required and where the water is treated, the required ultimate dimensions of the treatment units.

4.9.2 Variation in the Rate of Consumption

The average-rate of supply per capita is in fact the mathematical average taken over an average year.· Thus if Q is the total Quantity of water supplied to a population 'P' for 365 days, then the average rate of daily consumption 'q' is given by the following equation:

$$q = Q/(P \times 365) \text{ litre/capita/day.}$$

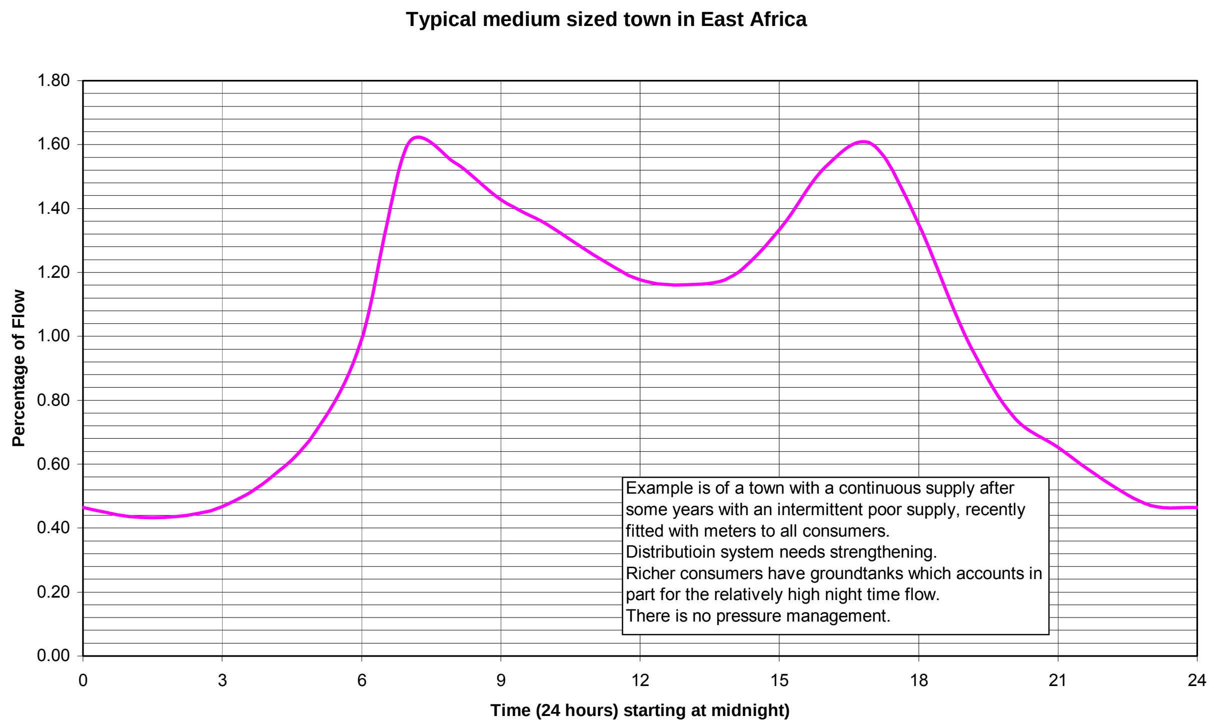
The types and nature of variations of demand of which 'q' is an average are given below.

Diurnal variation in Demand

The consumption of water is not uniform throughout the day. Generally two peak periods of demand are observed, one in the morning and one in the evening.

The maximum intensity of demand which occurs in the morning varies between about 1.5 and 2.4 times the average demand for the day depending upon the size of the of the population served (see Table 4.21).

An example of a diurnal variation curve for a medium sized urban area is given in the following diagram. In the absence of more specific information it may be used for centres with between 3,000 and 8,000 consumer connections.



Seasonal Variation in Demand

The further south one goes in Tanzania the more marked are the differences between the hot (summer) season and the cool (winter) season.

If the average value of consumption per day is considered separately for three seasons namely, dry summer, dry winter and rainy seasons it will be found that the average daily consumption during summer is about 35 to 50% more than in winter.· Similarly the daily consumption in the rainy season is 35% to 50% less than that in the dry seasons.

Combining the effects daily and seasonal variation it can be seen that the intensity of peak load in the period of maximum demand in some days in the summer dry season can reach an order of 2.0 to 2.5 times the yearly average value.

These variations must be considered when designing the various components of the water scheme.

Large impounding reservoirs are designed to cater for the variations and to ensure a steady delivery of water for a period from one drought year up to three consecutive dry years.

For the supply channels and mains for the conveyance of this water to the treatment works, a figure of between 1.35 and 1.50 times the average demand rate 'q' is used.

The pumping stations and treatment units are usually designed by taking 1.35 to 1.5 times the average demand rate since they are to meet the peak seasonal requirements. If the pumping is for less than 24 hours, then the above rate must be multiplied by the ratio of 24 hours to the hours of pumping.

Transmission mains are designed to meet peak daily demand whilst distribution mains in the supply area are required to meet peak period demand and in urban areas are designed for up to 2.5 times the average rate of demand.

Peak Factors

Though peak factors are necessary for calculating the actual peak period demand, application in individual cases is difficult.

The population data such as number of people using each domestic point, number of hours per day, people's habits in collecting is difficult to obtain and again may vary by season or by school calendar. Hence use of peak factors to calculate an individual's demand is impracticable and the figures given in Table 4.21 on the following page is but a rough guide to this.

However, peak factors must be adopted for the dimensioning of the various components in a water distribution network.

For every scheme, a study is required to establish the appropriate water demand and for large schemes it must be thorough.

For small rural water supply schemes a simpler method may be adopted to derive peak flows beyond the last storage tank with the peak flow for human consumption being given by the expression:

$$(\text{Average daily demand} \times 4) / 24$$

That assumes the peak demand is four times the average hourly demand or in other words the daily demand is drawn over 6 hours during the two peak periods of the day.

For livestock, it is usual to work on the basis of a ten hour period such that the peak flow may be taken as:

$$\text{Daily livestock demand} / 10$$

Thus in small rural water supply schemes providing water both for human consumption and livestock, the peak flow used for dimensioning of the distribution mains is the sum of the peak flow for human consumption and the peak flow for livestock consumption.

In many urban areas and particularly in those that have experienced past restrictions in water supply, a number of consumers and especially those in high cost areas and institutions etc., will have constructed their own ground level or below ground storage, often well in excess of their daily requirements. In such instances, the actual peak hour flow can be noticeably less than the figure of 2.5 indicated in the table.

In the absence of information to the contrary however, peak factors that may be used for different consumer categories are as follows:

TABLE 4.21: PEAK FACTORS FOR DIFFERENT CONSUMERS

CONSUMERS	PEAK DAY FACTOR	PEAK HR FACTOR	REMARKS
1. Residential			Should be based on study of population density housing type and water pressure tests in the area
– Low class housing	1.50	2.50	
– Medium class housing	1.30		
– High class housing	1.10		
2. Public Institution			In particular cases of ablution blocks of police lines and field force unit quarters a peak hour factor of 3.0 shall be adopted.
1. Prisons	1.10	4.00	
2. Primary schools	1.10		
3. Secondary schools	1.10		
4. Colleges	1.10	2.00	
5. Hospitals	1.10	2.50	
6. Dispensaries	1.10		
3. Commercial	1.20 to 2.30	1.80	
4. Industrial		1.80 to 2.50	Appropriate peak factors should be used for specific industries depending on factory sizes, process involved number or employees working hours, natural of reticulation system, if any based on industrial survey
5. Domestic Points	1.00	3.40	Constant flow or flow control device 10n 1/min for 12hrs n = number of taps (10 x n)/60 1/s
6. Livestock	1.00	2.0 – 3.0	

When it is impossible to separate consumers into categories as given in the previous table, but the total population of the area is known the table below can be used as a guideline.

TABLE 4.22: PEAK FACTORS

POPULATION	RANGE OF PEAK FACTORS	
	PEAK DAY FACTOR	PEAK HOUR FACTOR
10,000	1.80 - 1.50	2.40 - 2.0
10,000 - 30,000	1.50 - 1.40	2.0 - 1.70
30,000 - 100,000	1.50 - 1.30	1.70 - 1.60
100,000	1.30	1.60 - 1.50

4.10 WATER TRANSMISSION

4.10.1 General

In a water supply system, water has to be transported from source to consumers and usually through different stages by means of a variety of pipes made of different materials. Water can be piped by gravity or by pumping from an intake through a treatment process into storage reservoirs, from where it is distributed to the consumers.

The piping system has to be designed and dimensioned to meet the estimated water demand and pressure requirements, taking into account the possible implementation in stages.

The capability of a pipeline to convey water from one level, A, to a second, B, is calculated by a simple energy equation where:

$$E_A = E_B + h_f$$

where:

- E_A = the energy level at A,
- E_B = the energy level at B, and
- h_f = the friction head loss in the pipe between A and B.

The friction head loss, h_f is given by the equation:

$$h_f = i \times L$$

where:

- i = the slope of the hydraulic gradient along the pipe section A to B, and is a function of flow, inside diameter and roughness of the inside of the pipe and is calculated by using an appropriate formula as discussed in Section 4.13, and
- L = the total length of the pipe section from A to B taking into account the loss of head in both the pipes and the fittings installed in it as discussed in Section 4.13.2.

4.10.2 Dimensioning Flows

A forecast, on the water consumption in a water supply scheme, must be made for the given design period on the estimated water demand for all relevant types of consumption such as domestic, public, commercial, and industrial use as well as on the water requirements for the other purposes. The forecasts have to be detailed enough to give the water demand-by different pressure zones and consumption areas and should be given in order to give the basis for the design of different elements of the water supply system.

4.10.2.1 Average Daily Consumption

The average daily per capita consumption, (Q_{da}) is obtained by multiplication of the per capita consumption by the number of people. In urban areas this should be aggregated from the different categories or types of consumer. To this figure, a proportionate amount of the losses in the system have to be added and in any presentation the inclusion or otherwise of such losses should be clearly indicated. Net water demand excludes such water loss whilst total water demand includes provision for such system losses.

The average daily consumption figures are used mainly for economic calculations, to estimate revenue collection, to determine the price of water etc. They are normally not the dimensioning factor for the technical components.

4.10.2.2 Maximum Daily Consumption

The maximum daily consumption, (Q_{dmax}) is the average daily demand multiplied by the peak day factor (K_d) (See table 4.21). The maximum daily consumption is the dimensioning flow for the design of intake structures, treatment units, main pump-station and-rising or gravity mains which are to supply the whole service area.

4.10.2.3 Peak Hour Demand

Peak hour demand, (Q_{hmax}) is the highest consumption during a peak hour in a day.

It is obtained by multiplication of the average per hour consumption on a peak day by the peak hour factor (K_h). For the distribution network, the peak hour demand is in general the dimensioning flow. Nevertheless the capacity of the pipes must also be able to cater for the fire fighting water demand.

4.10.2.4 Fire Fighting Water Demand

Fire fighting demand, (q_f) has already been discussed in section 4.6.5.6. It is to be noted that water for fire fighting water is to be taken into account for urban areas only. It is to be assumed that during any fire there is an average day, peak hour, water consumption in the distribution network, ($k_h \times Q_{da}$)/24.

4.10.3 Choice of Dimensioning Flow

As a summary of the above point the following guidelines are given:

TABLE 4.23: DIMENSIONING FLOWS

COMPONENT	DIMENSIONING FLOW
Intake	$q_{dim} = Q_{dmax}/n$ where: n = number of operation hours
Treatment plant, Pumps* and Rising main	dim = dimensioning flow
Gravity main	$q_{dim} = Q_{dmax}/24$
Distribution main	peak hour demand
Secondary distribution	$q_{dim} = Q_{dmax}$
Check for Fire fighting demand;	$q_{dim} = q_f + K_h \times Q_{da}/24$
* **Pumps:** When pumping to a service reservoir the pumps are designed for peak day demand; & for peak hour demand when pumping directly to the distribution system	

4.10.4 Pressure Requirements

The design of distribution networks should be based on topographical information and if necessary a survey should be undertaken to confirm this and to map the supply area. The practicability of using aerial photography or archived satellite imagery to provide a background to any map should be considered.

4.10.4.1 Minimum Pressure

The residual pressure at any point of consumption should not be less than 5m head of water column during the peak flow situation in the network, whilst at fire-hydrants it should be 15 m.

Note: 10.0 m of water column = 1.0 kg/cm^2 ≈ 1 Bar
1.0 atmosphere = 10. 33m of water column = 1.033 kg/cm^2

Loss of head to a storied house within a plot can be calculated as follows:

TABLE 4.24: LOSS OF HEAD TO A STORIED HOUSE

PART	LOSS OF HEAD
Connection pipe to the house	1.0 m
Water meter	4.0 m
Inside piping	5.0 m
Total	**10.0 + n × 0.5 m**
Static head	n × 3.0 m
Residual head at the point of Consumption	5.0 m
Where n = number or stories	

The pressure requirement at the house connection is then: (15.0 + 3.5 × n) m of water column. This pressure has to be maintained during fire fighting water abstractions and the on-going average day/peak hour consumption in the network.

4.10.4.2 Maximum Pressure

To avoid bursts and leakages in the supply system as well as undesired noises and pressure shocks, the pressure in the distribution pipes at street level should not exceed 60m head.

When the pressure in the pipe exceeds the maximum allowed, the pressure can be decreased by using pressure reducing valves or break pressure tanks.

4.10.4.3 Pressure Management

Pressure management can be an important way of reducing real losses in a distribution system. It has been observed that in older parts of a distribution system in particular, there may be very little difference between day time and night time flows even though there is a marked reduction in consumption because increasing pressure simply increases the losses.

Provided due allowance has been made in the design and installation of the pipe, leakage loss in flexibly jointed distribution pipes has been shown to vary inversely as the pressure.

The relationship between pressure in Bars and physical loss in a distribution system may be approximated by the equation: Percentage loss = $(P/5)^{½}$.

Pressure(P) in Bars	5	4	3	2	1
Loss as a percentage	100%	89%	77%	63%	45%

This is why pressure management can be such an effective tool in loss reduction.

In such situations and instead of single stage pressure reducing valves (PRVs), two-stage or time-modulated pressure reducing valves supplying an area should be considered.

In the example given in Section 4.7.2, a theoretical assessment suggested that by supplying the Central Business District through time-modulated PRVs, with different night time and daytime pressures a reduction in % leakage from about 45% to 27% could be achieved. In practice and after installation of the time-modulated PRVs, the reduction was to about 30% but nevertheless very significant.

The only drawback to such an arrangement is in maintaining sufficient night time flows for fire fighting purposes but by judicious selection of controlled areas and fire hydrant siting this can often be overcome.

4.10.5 Pressure Zones

The service area having one and same top water level is called a pressure zone. A pressure zone is normally commanded by one or several high level reservoirs. The top water level of the reservoir is the reference level for the maximum pressure during low consumption and the low water level of the tank is the minimum reference level during the peak consumption.

If the town topography undulates much, then the town must be divided into different pressure zones, for example: low level, medium level, and high level zones. When the ground level varies more than 10m then zoning should be considered. The different elevations; and if the scheme does not permit construction of a number of reservoirs due to financial and other constraints, then separate distribution mains can be taken from the same service reservoir and each zone will have its own separate distribution system, but with interlinking between the zones via a normally closed sluice valve. During shortage of water these valves will be opened but throttled to give supply to the people both living in low level and high level areas.

The major running costs in pumping schemes are normally the energy for pumping. It is therefore very important that the pumping head .for the bulk of the water supplied into the scheme is as low as possible. If there are separate areas which will require high pressure than the rest of .the system it is normally cheaper to have a booster station for those areas or buildings than to lift the pressure zones. However, boosters should be kept to a realistic minimum because of the increased running and maintenance costs for every separate installation.

The pressure of the water supply should also be known by town planners and other authorities who are concerned in planning new residential areas and areas for other activities. This is to avoid future problems such as sudden infrastructural development at an elevation which cannot be served by the existing water supply at the time, due to lack of command to the area. Town planners and Engineers should foresee all these contingencies and plan properly in consultation with each other. The Engineer should always be responsible for deciding the zones.

4.11 WATER MAINS

4.11.1 Introduction

Pipelines are usually classified into different categories according to their function.

Trunk or transmission mains convey water over short or long distances, from source or treatment works to balancing storage or selected points in the distribution system. The following categories of trunk mains are considered:

(a) Gravity main

(b) Suction mains

(c) Rising or pumping main

The following classifications specify the physical working principles of the distribution system:

- Primary distribution mains are the pipes feeding the distribution network and are fed from storage or balancing tanks,
- Secondary, and in large networks, tertiary mains, collectively known as services mains are those pipes which connect the primary distribution mains to the individual consumers,
- Consumer connections are those pipes, preferably short in length, which feed individual consumer connections from the service mains, terminating at a stopcock and where fitted a consumer meter, and
- Plumbing pipes are those pipes within a plot and building which convey water to the various appliances.

4.11.2 Pipeline Wayleaves

For all pipelines it is important to obtain and secure a wayleave so as to avoid problems later on. Even in road reserves the alignment should be agreed with the road authority in advance and officially recorded so that even many years later there can be no argument when it comes to any dispute or compensation claim.

4.11.3 Water Hammer or Surge Pressure

Water hammer is a phenomenon that is liable to occur in any pressurised water pipeline and is set up by a sudden change in flow velocity such as occurs when stopping or starting of pumps or closing or opening. Pump stopping and valve closure have the more severe effects.

This sets up periodic pressure oscillations which move backwards and forwards along the pipeline. The water column in the pipeline takes a brief time to adapt to the new conditions and during this period of adapting to the change different parts of the water column can behave differently, and can separate and come back together again causing rapid pressure fluctuations.

Water hammer can degrade the integrity of a pipeline system in one of two ways: through catastrophic failure or by benign failure. Catastrophic failure occurs during a water hammer event when pipes rupture, joints move, excessive noise occurs, etc. Benign failure may occur over a period of years and may consist of lining failure and pipe wall pitting.

Designers must be aware of the problem of surge and take it into account when designing the pipe lines in a water supply system.

Water hammer effects are usually the most severe in pumping mains upon stopping of a pump when the flow ceases quite abruptly causing a negative pressure wave to travel up the pipeline to some end feature before returning again as a positive wave. However water hammer does occur in gravity mains and in distribution systems and always must be allowed for in pipeline design and with thermoplastic pipes often requires increasing the wall thickness and hence the pressure class to deal with.

Because the profile of a pipeline has a significant influence upon the effects of water hammer, and pipeline high points in particular, means of containing such effects at these points are essential and one of the reasons why air valves are so important. The water column will tend to separate at high points during a negative surge wave, and even water vapour vacuum conditions can result for a short period. Upon the return of the pressure wave, the water columns collide with extremely high and even catastrophic pressures resulting. The topic of water hammer is dealt with further in Section 4.11.7.

4.11.4 Air Valves and Washouts

Air valves should be fitted at all high points and at significant changes in downward slope and washouts should be fitted at low points. Even in flat areas an air valve at every 600 m to 1000m is necessary as air bubbles form as water pressures fall. To help prevent the formation of air pockets, minimum slopes should be 0.3% for DN ≤ 200 mm and 0.2% for DN > 200 mm.

Air valves are required to vent any air bubbles that are conveyed or formed in the water as the development of air pockets at high points can greatly reduce or even stop the flow of water. They are also required to vent large quantities of air when pipelines are filled and as noted above to help deal with the problems associated with surge. This third function of an air valve has been all too frequently ignored in the past.

Suitably sized air valves should be located at upturned tees at all high points fitted with an isolating valve and it is recommended that at all high points in a pipeline air valves should be of the triple-function anti-surge anti-shock type that have been designed using controlled air transient technology (CATT). The air valve tee should be designed as an air accumulator tee with the initial tee branch 0.6 times the main pipe diameter. This is then reduced further to the diameter of the air valve. Use of such air accumulator tees and the CATT type of air valve, provided they are maintained in an operational condition, may actually eliminate the need for other surge protection devices usually installed at pump station outlets.

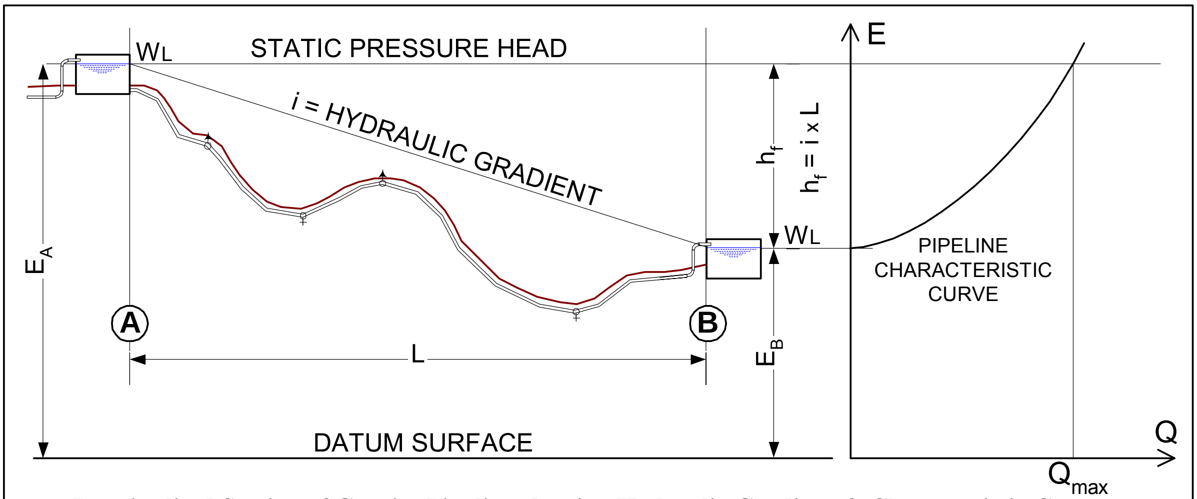
Washouts are required at low points so as to be able to periodically flush out the pipeline to help remove any matter that tends to accumulate at such points. Periodic flushing is essential because the matter that accumulates will include organic matter and over time this will turn the accumulation septic. If then disturbed this causes a ‘plug’ of foul water to be conveyed onwards that may be beyond the ability of the residual chlorine to disinfect before reaching the next consumer draw-off point.

Like air valves, washouts are not the same diameter as the main, and for washout tees the empirical formula used is ½ diameter of main + 25mm. For large diameter mains the washout tee should be an invert tee so as to be able to help evacuate the water and any settled deposits.

4.11.5 Gravity Mains

A gravity supply is of course the most preferable in respect of economy in construction, operation and maintenance. The main should always be of such a diameter that the total quantity required for the future projected peak day demand can flow through the pipe in 24 hours.

With reference to the Energy Equation quoted at the beginning of Section 4.10, the situation in a gravity main is illustrated in the Figure below:



Longitudinal Section of Gravity Pipeline showing Hydraulic Gradient & Characteristic Curve

The gravity main should be as far as possible be on a constant falling gradient, avoiding high points and low valleys. Where the static pressure exceeds the allowable pipe pressures, a break pressure tank with ball valve should be installed. Excessive high points should be avoided and on no account should the pipeline be laid higher than the hydraulic gradient as this results in negative pressures.

As far as practicable, rock outcrops should be avoided, as drilling and blasting a trench in rock is very expensive. The pipeline should be aligned around the rocky areas either by using large radius bends or better still by using the slight deviation angle at each pipe joint, staying within the limits which depend upon the pipe joint but are within the range of 5° for small diameter pipes to 2° for larger diameters, but always as prescribed by the manufacturers. However, some designers prefer to limit deflection at rubber ring joints to 2^{o} for all diameters to reduce the risk of leakage.

Where it is unavoidable for the pipeline to be laid above ground, the pipe needs to be fixed freely on concrete supports and hold in place by metal ring brackets, which are set into the concrete support. The pipe needs to have free movement 'within the ring bracket to allow for expansion and contraction. Pipes above ground can only be of barrier coated steel or ductile iron (D.I.) or in exceptional circumstances and in diameters less than DN80 in galvanized steel (GS). The support pillars should be placed at intervals equal to length of the pipe.

Concrete anchors, also termed thrust blocks, should be constructed at every 200m on all gradients and at much lesser distances on steep gradients and still lesser distances on steeper gradients. Except on welded joint steel pipelines, thrust blocks should be provided at all horizontal and vertical changes of direction at all equal tees and at all valves. However, on large diameter, high pressure, welded joint steel pipelines the pipe manufacturer should be consulted in this regard to ensure that they have allowed for the expected forces in their design. The concrete thrust block should be formed so that it follows the curvature of the pipe. The pipe should however never be fully encased by concrete.

Where tie-in bends and tees are to be inserted into live systems that can only be shut down for a short period it may be necessary to first cast a thrust wall parallel to the live main and initially transfer thrust to this thrust wall by a horizontal steel joist fitted with a curved thrust transfer plate fitting snugly to the new bend or tee. This thrust transfer joist is then cast into concrete to complete the thrust block as soon as the tie-in connection is completed and tested for water tightness and the system is put back into operation.

Non-return or reflux valves can be located at distances of 3 km to 5 km to facilitate maintenance and repair and in addition they will help in reducing water hammer.

For the purpose of inspection, maintenance and replacement, unions on small diameter and flanged joints with flange adaptors on medium and larger mains should be considered every 350 m to 500m.

Where sluice or gate valves are installed in the main, care should be taken in locating these section valves in respect of high pressures and water resulting hammer due to the closing and opening of these valves.

Gravity main may be of D.I., boundary coated steel, or thermoplastic material including PVCu and PE_{100}. Wall thickness shall be based on the design procedures laid down in the subsequent sections of this Chapter of the Manual but in general be sufficient to withstand both the actual working pressure in the pipes as well as the requirements of the site location and any vacuum, surge or water hammer risks. Asbestos cement pipes shall NOT be used for domestic water supply.

For small diameters where D.I. and boundary coated steel is not available and where thermoplastic pipes are unsuitable, then and only then may GS pipes be considered. To give them a satisfactory working life, GS pipes laid on or below ground may require additional corrosion protection, especially at threaded, socketed joints. In exceptional circumstances but only for trunk mains without consumer connections, Glass Reinforced Plastic (GRP) pipes may be considered if, for example, this is required by a Project Financier.

The design and use of GRP is not covered in this Manual and requires special design and installation attention by use of internationally acceptable Design and Installation Specifications. For all pipe wall materials, pipe manufacturer's own design and installation information should only be used where the Designer can satisfy himself that this meets international requirements and is supported by a copy in English of the appropriate standards, internationally recognized design manual and codes of practice. In such circumstances, manufacturers should be required to provide a specific worked example of the design calculation to support their offer.

Special care should be taken where mains cross beneath roads and tracks, where a minimum of 1m cover should be provided. It is an advantage when the pipes can be laid inside a cylindrical sleeve of reinforced concrete, steel, D.I. or C.I. pipe. These cover and protection of these should extend 3m beyond the width of the road at either side.

In general, gravity mains should be designed so that the available pressure head is just lost in overcoming the frictional resistance to the flow of water.

The velocities to be generated are therefore so maintained that they are (a) neither too small to require a larger size diameter pipe, nor (b) too high to cause excessive loss of pressure head.

4.11.6 Suction Mains

As far as practicable, suction mains should be designed based on the following principles: -

(a) Except in the cases of wet well pumps, and pumps with a positive suction arrangement, each pump should have a separate suction line fitted with strainer and a non-return foot valve. This is so as to avoid air entrainment into the suction line of a duty pump from a pump that is out of service or shut down.

(b) The joints in the suction line shall preferably be flanged and appropriately bolted and fitted with proper water sealing liner material such as rubber gaskets.

(c) All suction mains with a positive suction head should have a sluice valve immediately before the pump.

(d) The suction pipe should have a continuously rising slope towards the pump inlet branch so as to eliminate all possibilities of air pockets being trapped in the suction pipework. Sharp bends should be avoided and long radius bends be placed at the appropriate points.

(e) The diameter of the suction pipe should be sized according to the flow and allowable head loss and therefore may not be the same diameter as the pump inlet itself. As far as possible it should be of a larger diameter or be designed for a very low velocity in order to reduce friction losses and the drawing in of sediments

(f) A short straight pipe adjacent to the suction branch is desirable. Where a bend is unavoidable it should be of the largest possible radius.

(g) The suction line should be accessible and not therefore embedded in concrete.

(h) Foot valves, strainers bends etc. must be selected such that they will provide minimal restriction to flow.

Such a selection shall be done with critical consideration of the following formula: -

$$P_a = (P_v + h_{fs} + h_{s1})\ m + (NPSH_R + 0.5)\ m$$

Where,

P_a = atmospheric pressure (this is not constant). At sea level P_a = 1 atmosphere = 10.33m, but is lower the higher the altitude. See Table 4.25 below:

Pv = vapour pressure at operating water temperature. See Table 4.26 below:

h_{fs} = head loss from all causes in the suction line,

h_{sl} = permissible suction lift,

$NPSH_R$ = net positive suction head required,
= P_a - (P_v + h_{fs} + 0.5 + $NPSH_R$) = h_{S1}, (net permissible positive suction head)

0.5 = recommended safety margin

TABLE 4.25: ATMOSPHERIC PRESSURE VS ALTITUDE

ALTITUDE (Metre)	P_a	BAR Atm	COLUMN of WATER (Metre)
0	101,300	1.013	10.13
500	95,500	0.955	9.55
1,000	89,900	0.899	8.99
2,000	79,500	0.795	7.95

TABLE 4.26: TEMPERATURE VS VAPOUR PRESSURE

TEMPERATURE ^{0}C	P_v METRES
0	0.062
10	0.125
20	0.238
26	0.343
50	1.258

As a result, the practical suction head is usually as indicated below: -

TABLE 4.27: PRACTICAL SUCTION HEAD (REPRESENTATIVE FIGURE)

ALTITUDE ABOVE MEAN SEA LEVEL(METRES)	PRACTICAL SUCTION HEAD (METRES)
0	5.0
500	4.5
1,000	4.0
1,5000	3.5
2,000	3.0

The permissible suction lift must therefore be limited taking into account all the information on all factors shown in the formula above. Based on the above, the following is given as a guide.

(a) The suction inlet must be designed carefully. Ideally, the pipe should be at least 5 pipe diameters below the surface and at least 0.5m from any side wall. If the pipe must be at a depth significantly less than this, it is recommended to be positioned within 150mm of the side wall to reduce the possibility of vortex formation.

(b) Where possible the suction arrangement should be such that the pump can operate with a positive suction head at least at start-up, i.e. the suction sump (tank) could be above the pump floor level.

In this case, a common suction line or manifold can be adopted but each pump should have an isolating valve immediately before its suction branch to facilitate maintenance. Each suction branch shall be designed to form not more than a 45° angle with the common suction line to ensure laminar flow towards the pump.

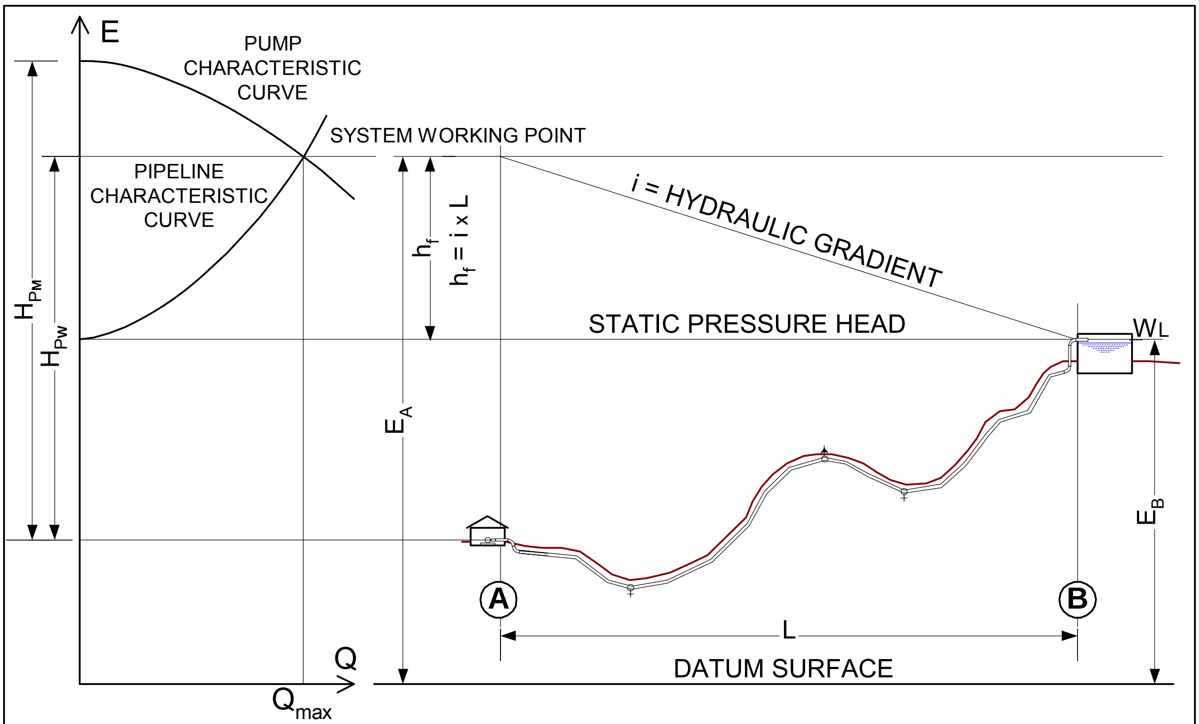
(c) Where negative suction head is in avoidable, i.e. with a suction sump below the pump floor level, no attempt shall be made to incorporate an isolating valve in the suction line. It should be noted that such a provision is unnecessary and in fact dangerous to the life of the pump since there is a possibility of the pump being operated with the suction line completely shut down.

(d) A priming arrangement shall be included by providing a bypass from beyond the main system non-return valve to the pump delivery branch on the pump side of the individual pump non-return valve

4.11.7 Rising or Pumping Mains and Surge

A rising main should where possible be sized such that the total quantity required daily for the projected population is able to pass through the pipe in not less than 16 hours.

With reference to the Energy Equation quoted at the beginning of Section 4.10, the situation in a pumping main is illustrated in the Figure below:



Longitudinal Section of Pumping Main showing Hydraulic Gradient & Characteristic Curve

Pipe materials for rising mains should preferably be restricted to D.I. or Steel whilst PVCu should not be used. Special care should be taken to see that the wall thickness of the pipes are able to withstand the actual pressure, the highest pressure occurring at the lowest point and the lowest pressure at the highest point both inclusive of water hammer surge.

Otherwise all other points as laid down for gravity mains are equally valid for rising mains.

Where no other alternative exists, a service connections may be taken from a rising main with due regard to the pressure in the main at the takeoff point. In such cases the insertion of a constant flow valve at the domestic point supplied by this service connection is necessary. However wherever practicable and especially where a small community is to be supplied the supply should be into a small balancing tank from where the service main is then connected.

A reflux (non-return) valve must be connected as close as possible to the pump, but subject to the manufacturer's recommendations regarding minimum distance, followed by a valve in the case of a centrifugal pump. If throttling is required, the valve should be a gate (sluice) valve; however where the valve is only for isolation purposes then a butterfly valve can be considered. A reflux valve must always be fitted when the discharge head exceeds 10 metres. When selecting the most appropriate type of reflux valve, consideration must be given to any water hammer effects, especially if a slam-tight reflux valve is selected.

On no account should a valve be installed where a reciprocating (piston) pump is being used. In that case, only a reflux valve should be installed

It is preferable that an additional reflux valve should be installed on the common manifold rising main immediately after the pump house. This then provides the main source of protection against hydraulic surge. In addition to a reflux valve, surge protection equipment on the system may be necessary on the rising main outside the pump house.

Where a rising main is fitted with CATT type air valves such surge protection equipment may be unnecessary but this needs to be confirmed by a surge analysis. In all cases and even when a specific surge protection device is found to be unnecessary, a burst disc device should be installed so that should any of the CATT air valves or other equipment go unserviceable, a final protection to the system is provided.

Manufacturers of CATT types of air valves will often undertake a surge analysis without charge where the intention is to specify their equipment. Where surge protection equipment is shown to be needed, this should be located beyond the reflux valve on the common manifold. The type of protective equipment shall be determined following a proper and comprehensive surge analysis in accordance with procedures laid down under protection against water hammer or hydraulic surges in Chapter 5 on Pumping Plants and Pumping Systems.

To avoid complicated hydraulic surges, apexes on a rising main shall as far as possible be minimized if they cannot be avoided all together. However the best profile for a rising main is a concave or increasing grade shaped profile type along the longitudinal section of the rising main as far as avoidance of sub-atmospheric pressure is concerned.

The flow velocity in a main (average-flow velocity of the pumped medium) shall be kept as low as economically possible as the surge pressure approximates to a direct function of the flow velocity. Other important factors are pipeline profile and pipeline length.

An initial check on likely maximum surge for a single circular pipe without diameter change along its length, L, can be made using the expressions

$$\Delta H = \pm V \times c_p / g$$ where:

V = average flow velocity, before rapid stoppage or closure in m/s,
c_p =surge wave velocity; which normally ranges between 600 - 1200 m/s depending on a number of factors including physical characteristic of the main and the pumped medium, and is equal to $c_w / (1 + (E_w / E_p) \times (d / t))^{0.5}$, where

c_w = celerity (speed) of the pressure wave in a column of water – 1425 m/s
E_w = the bulk modulus of water =2230 N/mm^2 at 30^{o}C
E_p = modulus of elasticity of pipe wall material in N/mm^2,
g = acceleration due to gravity (9.81 m/s^2),
d = mean diameter of pipe in metres, and
t = pipe wall thickness in metres, and

The values of E_p shown in Table 4.28 may be adopted for different materials.

TABLE 4.28: MODULUS OF ELASTICITY FOR DIFFERENT MATERIALS

MATERIAL	Ep in N/mm$_2$
PE_{100}	600-700
PVCu	2,800
Concrete	28,000
Reinforced concrete	31,000
Prestressed concrete	35,000
Ductile Iron pipe	170,000
Steel	207,000

In a case where the pump stoppage time or valve closure time is greater than the time it takes the wave to travel forwards and backwards (the reflection time), then the value of ΔH is less, where

the reflection time, $T_r = 2 \times L/c_p$

Abrupt pump stoppage can be avoided by fitting a pump with a heavy flywheel designed such that the pump still has some rotation at the end of the reflection time. There is a type of flywheel that is oil filled and can be adjusted to rotate the pump at a speed equal to or slower than that of the motor. This can be useful during the early years of a scheme when demand is les than maximum as it avoids the need for expensive variable speed motors. It has the added advantage that it can considerably reduce the magnitude of the surge wave.

Where ram pumps are installed, a pressure relief valve shall be installed on or after immediately the pump and before the valves mentioned above unless arrangements have been made by the manufacturer.

The likely significance of the effects of surge in Pumping Mains where centrifugal pumps are involved can be checked initially by considering the following 12 questions [< >]:

1. Are there any high spots in the profile of the pumping main where there is the risk of the occurrence of a vacuum which could cause a parting of the water column when a pump stoppage occurs?

[< >] Steel Pipe – A Guide for Design and Installation, Manual of Water Supply Practice M11 AWWA, 2004

2. Is the length of the main less than 20 times the head on the pumps (both expressed in the same units)?
3. Is the maximum flow velocity in the main in excess of 1.2 m/s?
4. Is the safety factor of the pipe less than 3.5 (related to ultimate strength) for normal operating pressure?
5. What is the natural decreasing rate of the water column when a pump shuts down? Will the column come to rest and reverse its direction of flow in less than the critical surge-wave time for the main?
6. Will the NRV at the pump station close in less than the critical time for the main?
7. Are there any quick-closing automatic valves set to open or close in less than 5 sec?
8. Would the pump or its driving motor be damaged if allowed to run backwards, reaching full speed?
9. Will the pump trip before the discharge valve is fully closed?
10. Will the pump ever be started with the discharge gate valve open?
11. Are there booster pumping stations in the system dependant on the operation of the main pumping station?
12. Are there any quick-closing automatic valves used in the pumping system that would become inoperative with the failure of the pumping system pressure?

If the answer to any of these questions is ‘yes’ there is a strong possibility that serious surge problems will occur if suitable surge alleviation devices are not included in the system. If the answer to two or more questions is ‘yes’, surges will probably occur with severity in proportion to the number of affirmative answers unless suitable surge alleviation devices are included in the system.

All rising mains passing through walls of building or through structures shall be independent and free of the wall.

The pumping head is the total head, which is the static head plus the friction head loss for the design flow rate. The pump selected must be able to provide this head (see Figure in Section 4.11.7).

Pump head = static head + the total pressure losses in (m).

This calculation should be repeated for several pipe diameters. Each combination of pumping head and pipe diameter· would be capable of supplying the required flow rate of water over the distance. However, only one pipe diameter is likely to represent the least cost choice taking into account the initial cost (capital investment) and the energy costs for pumping.

The total cost, capitalised, is then the basis for selecting the most economical pipe diameter.

4.11.8 Distribution Mains

The primary distribution main is defined as the pipeline which feeds a zone within the distribution network and itself is supplied from a balancing or storage tank. All points discussed above on gravity mains are equally valid for the distribution system.

In the distribution main it is good practice to install gate valves (instead of reflux valves as in the rising main) every 3km or so to facilitate easy maintenance and repairs. It is preferable to incorporate these with the washout arrangements.

As far as possible no service connection shall be taken from the primary distribution mains, however in smaller supply systems, adherence to this ruling will not always be possible.

The primary main system then supplies zones with secondary and in large networks even tertiary distribution mains. These then supply water to individual consumer connections through service mains. As a general rule, the secondary distribution network will be fitted with fire hydrants whilst any tertiary system may not.

Each zone should be supplied through one or more bulk meters for measurement and evaluation of losses and possible upward trends in such losses.

Wherever possible the distribution mains in a zone shall be in the form of a ring, with valves on the branches immediately after the tee. Other valves as necessary for the satisfactory control of the distribution system shall be installed in the ring main.

4.11.9 Service Mains and House Connections

Service mains are defined as those pipes in the distribution systems which supply the individual service connection to properties.

Where a service main draws water directly from a primary main, a valve shall be placed straight on to the branch of the tee. Similarly where one service main branches from another the branch shall be provided with a valve.

All points discussed for gravity mains are generally valid for service mains. Blank ends to services mains should be avoided where practicable.

4.11.9.1 Domestic Points (Community Taps and Kiosks)

To achieve the water sector MDGs, Domestic Points (DPs), either community taps or water kiosks, are essential components of any new or expanded water supply. In peri-urban or informal urban housing areas this should as far as possible be water kiosks. In rural areas the decision on whether to install a community tap or kiosk should be taken by the community but only after they have been fully informed on what this means. In either case, water should be charged for on some volumetric basis.

Because DPs play a key role in the supply of water in rural areas and to the urban poor and will continue to do so for many years to come, the most appropriate ways of supplying water through them are under review worldwide. The following guidelines should not therefore be rigidly followed but used as a basis from which Designers can start.

Regardless of the detail, involving the community in finding proper sites for the DP has proven to be an efficient way to improve psychological ownership, prevent vandalism and make treated water accessible.

Present guidelines are:

- The placement of DPs must take the preferences of the operator or attendant and of future customers into account, as well as the technical and commercial constraints and objectives identified by the designer. In other words, a domestic point should be placed in a way that it can serve a maximum number of customers in an efficient and customer friendly manner.

- Provision should be made for a supply of 25 l/p/d unless there is evidence that at the price to be charged, demand is likely to exceed this in which case and if the water source permits, the higher figure may be used.
- The minimum static head should be not less than 5m and the maximum about 25m. The flow from each tap at a DP should not be less than 10 l/min, and where appropriate, a constant flow valve should be installed.
- Where water is only available intermittently or supply pressures are low provision of storage should be considered.
- No one should have to carry the water more than 250m in an urban area and 400m in rural areas whilst the time spent on collecting water should not exceed 30 minutes.
- A charge should be made for the supply of water from a DP. This should preferably be based on a coin still in wide circulation per container of 20 litres (e.g. 5/- or 10/-).
- A single DP should in general not supply more than 200 households, or 1,000 to 1,800 persons, which number is likely to be the minimum if the supply is to be financially viable for an operator or a community.
- There should be one tap outlet for every 250 persons or 50 households supplied.
- In urban areas the operator should be selected or appointed by the household representatives and the street executive officer [< >].
- In urban areas, the supply is best provided through a formal structure, and the operator should be encouraged to sell other non-contaminant dry goods items both to augment income and extend operating hours.
- The siting of DPs should be so that the area can be kept clean with minimum effort and there must be sufficient scope for natural drainage of waste water. Where a clothes washing slab is provided this should be not more than 20 metres from the water point.
- The operator or attendant should be made responsible for keeping the area clean and drained without the formation of any stagnant water pools.

4.11.9.2 Individual Connections

All institutional and industrial connections should be considered in the Design.

All individual connections shall consist of a pipe of minimum 15mm diameter which should be taken from the nearest distribution pipe by a ferruled saddle or reducing tee, depending on the size and material of the distribution pipe. Ferrules alone should not be used as experience has shown these to be a major source of leakage from a distribution system. The services pipe will be laid till just outside or just within the plot boundary, where the stop cock and meter will be located. The locational preference should be advised by the Client as there are advantages and

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< > Every ten households constitute an administrative unit. Once every five years each 10 households elect its leader who is known as the Ten Cell Leader or *Balozi Nyumba Kumi*. Ten such leaders work with Muku *WA Muta* or street executive officer that is appointed by the government to supervise the social development activities of the street.

disadvantages to both alternatives. Connections to public institutions should be included in the design. Pressure will be the same as for DPs.

For high rise buildings, customers must be prepared to provide their own pumping equipment. However this should never draw directly from the service pipe but from a balancing tank which should also be provided by the customer. Such tanks shall be provided with a ball or float valve.

If it is the Clients policy, and to prevent wastage, a constant–flow valve should be installed on each service connection.

The cost of labour and materials for the service connections shall be charged to the customer or property owner, except that the meter shall not be charged for, but remain the property of the water authority who may nevertheless charge a monthly meter rent.

New customers should not be allowed to purchase their own service connection materials as use of sub-standard components are then common. Instead the water authority should stock the required materials that meet their standard requirements, and these then onward sold to the new customer. Alternatively and where a water authority is happy with the quality of materials stocked by a particular local supplier or suppliers, such suppliers can be specified with proof of purchase then accepted by the authority.

4.11.9.3 Stock Watering Troughs

All stock watering troughs should be provided, with a ball or float valve. A separate gate valve housed in a valve box should be provided in the service pipe to the troughs well outside the herd standing area.

A standard trough can accommodate 1,000 head of cattle over a 10 hour day. The service pipe should be able to take a flow equal to the total daily demand of livestock divided by 10.

Troughs should be situated on elevated ground so that adequate drainage can be arranged. In specific cattle areas the distances between troughs should be not more than 6 km. However the cattle trough and any cattle dips should be located well away from DPs.

Livestock demand should not be considered in urban areas or where pastoral land is scarce.

4.12 WATER DISTRIBUTION SYSTEMS

4.12.1 General

The purpose of a distribution system is to convey a sufficient quantity of potable, clean, and pressured water within allowable limits. Distribution systems normally account for about 40-70% of the investment costs. Therefore it is essential to carry out proper designs and layouts of the distribution system. The main criteria for the distribution system may be classified as:-

(i) Functional

(ii) Hydraulic

Functional aspects include proper layout of pipes, reservoirs and booster pumping stations, selection and proper location of valves, specials, meters etc. for efficient operation and maintenance and overall economy in cost.

Proper hydraulic consideration will facilitate adequate residual pressure during peak demand and the meeting of fire fighting requirements.

4.12.2 Alternative Distribution Layouts

There are principally four systems adopted for the layout of pipelines to distribute water through mains, sub-mains, branches and service pipes. Each method of layout has its own advantages and is suitable under specific conditions.

4.12.2.1 Branch or Dead-end (Tree) Systems

In general, a branched system is adopted for small capacity community supplies delivering the water mostly through DPs and having few if any house connections. This system is also applicable in the case of towns that have developed in an irregular manner such that there is no regular layout of roads and for small water supplies. This arrangement has the advantage that the design is straightforward and very simple. Both the direction of flow and the flow rate can be readily determined.

The disadvantage of this system is the large number of dead ends, stagnation of water leading to contamination, and the isolation of a large area of supply during repairs.

4.12.2.2 Grid Iron Systems

In this system, the layout of the pipelines assumes the shape of a network and dead ends are kept a minimum by inter-connection. This system facilitates any one point being fed at least from two directions and the water is kept in circulation. This system is useful for towns that have a rectangular road layout. In case of emergency, water can be a drawn from more than one direction.

The disadvantage is that the hydraulic computation is more difficult and for all but the simpler systems requires the use of computer software. In addition, the system requires a larger number of valves and fittings. A looped network usually has a ring of mains to which secondary pipes are connected. In large (urban) distribution systems the secondary pipes are themselves usually interconnected which requires many valves and special fittings.

For small distribution systems, the over crossing of secondary pipes that are not interconnected may be advantageous as it may result in a considerable cost saving.

4.12.2.3 Ring Main (Circular) Systems

In this system the supply main is laid around the zone or district of distribution. The flow is directed and reaches any location from both ends.

4.12.2.4 Radial Systems

This is the opposite of the ring system. The distribution reservoir is placed at the centre of a district or zone and the supply pipes radiate out away towards the boundary of the area. If the town has a radial layout of roads, this method may be the most suited. Radial systems ensure a quick satisfactory service and pipe diameter can also be easily computed.

4.12.3 Design Requirements for Branch Systems

The design of distribution system in a branch or dead end system is very simple. The procedure is as follows:-

i. Detailed maps are prepared for each area showing the streets and the blocks of houses, parks etc.

ii. A tentative layout of the distribution pipes is then drawn along the streets and the distribution of flow is marked by arrows.

iii. The position of sluice valves, fire hydrants etc. are marked.

iv. The population to be served by each section of the pipe length between two sluice valves and between junctions or branching off points is noted against each section.

v. Knowing the unit rates of demand, the quantity of water to be supplied in each section can be determined.

vi. Then starting from the farthest end and proceeding towards the distribution reservoir or the pump house, the total quantity to be carried by each section of the pipe length can be summed on a cumulative basis.

vii. This gives the average flow the pipes have to carry whilst the maximum flow will be 2.5 to 3.0 times the average. This depends on the supply position also. If the town is supplied intermittently only, the maximum flow rate would be 4 to 5 times the average rate. To this flow, the fire demand and other special requirements should then be added.

viii. The pipe diameters are initially calculated assuming that velocities of flow generated in the pipe lie between 0.6 to 2.00 m/s and that they are lower in smaller diameter pipes and higher in the larger pipes. The recommended velocities are:-

TABLE 4.29: GUIDE TO DISTRIBUTION VELOCITY VS DIAMETER

PIPE DIAMETER RANGE (mm)	NOM. DIA. (mm)	VELOCITY RANGE (m/s)	FLOW MIN (l/s)	FLOW MAX (l/s)
Small, DN 50-110	50	0.60 - 1.00	1	2
	65		2	3
	80		3	5
	100		5	8
	110		6	10
Medium, DN 125-250	125	1.00 - 1.50	10	18
	150		18	30
	200		30	45
	225		40	60
	250		50	80
Large, DN 300-500	300	1.20 - 2.00	80	140
	350		120	190
	400		150	250
	450		190	300
	500		250	400
Because internal diameters differ depending on pipe, these values should be used only as an initial guide, the selection to be reviewed once the pipe material is known.				

For larger pipe diameters the manufacturers recommended values should be taken.

Based on the flow and velocity criteria, the tentative pipe sizes are selected. Thus the pipes are selected for the first round trial.

Once the diameter of the pipes, the velocity of flow and the length are known, the head lost due to friction in the pipe can be found by a suitable friction loss formula. Because applying this formula and getting the friction loss in every section of a large distribution system is very tedious and cumbersome, discharge tables showing friction losses can be used or preferably a computer spreadsheet. If the latter is used then the preferred formula is that of Colebrook-White as a simple iterative procedure built into the spreadsheet will perform the calculation.

Thus after getting the head lost due to friction in various pipe lengths, the residual pressures remaining throughout the network can be calculated by deducting these from the initial pressure head available at the distribution reservoir. These should be sufficient for the water to reach the highest and/or farthest point of supply with the minimum residual pressure. If this is not achieved then a next larger diameter in any salient section should be assumed. If the residual pressure is very high than pipe diameters in those and preceding sections should be decreased so that a combination is achieved for an economical system. Several trials in computation may be necessary to achieve this.

A final computation is then required once the detailed design of the system is nearing completion, and particularly where thermoplastic pipes are included, the final wall thickness have been determined.

4.12.4 Design Requirements for Grid Iron Systems

In grid iron systems, water flows and reaches different points via more than one route and hence first of all, the flow quantities going via each route need to be found. Once these are known the design can be carried out as explained above.

To carry out such an analysis for all but the very simplest of networks by hand is very time consuming and even in a network of no more than 12 pipes is best undertaken by spreadsheet or better still by purposely developed hydraulic analysis software.

Such pipe network calculation uses the steady state energy equation, Hazen Williams or preferably Darcy Weisbach friction losses, and the Hardy Cross method to determine the flowrate in each pipe, loss in each pipe, and node pressures. Minor losses (due to valves, pipe bends, etc.) can be accounted for by using the equivalent length of pipe method.

The Hardy Cross method which dates from 1936 is also known as the single path adjustment method and is a relaxation method. The flowrate in each pipe is adjusted iteratively until all equations are satisfied. The method is based on two primary physical laws:

1. The sum of pipe flows into and out of a node equals the flow entering or leaving the system through the node and
2. Hydraulic head (i.e. elevation head + pressure head, Z+P/S) is single-valued. This means that the hydraulic head at a node is the same whether it is computed from upstream or downstream directions.

Pipe flows are adjusted iteratively using the equation opposite until the change in flow in each pipe is less than the convergence criteria:

$$\Delta Q_i = \pm \sum_{i=1}^{\#\text{Pipes in Loop}} H_i \Bigg/ \left[n \sum_{i=1}^{\#\text{Pipes in Loop}} \left(\frac{H_i}{Q_i} \right) \right]$$

Numerous software packages exist across a wide price range, but for Designers who do not otherwise have access to such software, the EPANET 2 package is suggested.

EPANET was developed by the Water Supply and Water Resources Division (formerly the Drinking Water Research Division) of the U.S. Environmental Protection Agency's National

Risk Management Research Laboratory. It is public domain software that may be freely copied and distributed.

The Windows version of EPANET provides an integrated environment for editing network input data, running hydraulic and water quality simulations, and viewing the results in a variety of formats. These include colour-coded network maps, data tables, time series graphs, and contour plots. For those who wish to work through a simple example, reference should be made to the 1997 edition of this Manual, section 4.8.3 starting on page 164.

4.13 HYDRAULIC COMPUTATIONS

4.13.1 Full Bore Flow in Pipes

The Hazen-Williams and the Colebrook-White formulae are the two most widely used for determining the flow in pipelines. Prior to the advent of spreadsheets and because of its simplicity, the empirical Hazen-Williams, formula was favoured for calculating flows in raw and potable water pipelines and the iterative Colebrook-White equation often reserved for the determining the hydraulic performance of sewage, drainage and effluent systems, designers then tending to rely on nomographs that gave only approximate results. Today, the Hazen Williams formula should only be used for a quick preliminary assessment when access to a spreadsheet version of Colebrook White is not available.

The **Hazen-Williams** formula has the advantage of simplicity and for determining the flow of fully turbulent raw or potable water at normal temperatures will give results of sufficient accuracy for all initial purposes and where only a hand held calculator is available.

The formula is not a homogeneous expression but includes a coefficient that has units. As a result when working with a calculator extreme care must be taken to use the proper units for all of the inputs. In SI units the formula can be conveniently expressed as:-

$$V = 0.457 \times 10^2 \times C \times D^{0.63} \times i^{\,0.54}, \text{ or } Q = 0.239 \times 10^5 \times C \times D^{2.63} \times i^{\,0.54}$$

Where:

$Q = A \times V \;=\; (\Pi \times D^2/4) \times V$

A = Internal cross-sectional area of the pipe

V = Velocity in m/s

Q = Quantity in l/s

D = Pipe internal diameter in mm

I = Hydraulic gradient (dimensionless)

C = Hazen William friction coefficient (dimensionless)

By selecting the appropriate value for the coefficient C, the Hazen – Williams formula can be used for all types of pipe materials. Table 4.30 shows suggested values of C for various pipeline. Values shown are based on case studies and consequently account is taken of losses due to irregularities at the joints. Upper values are for new pipes and lower values for older pipes.

TABLE 4.30: RECOMMENDED VALUES OF C IN HAZEN WILLIAMS FORMULA

CONDUIT MATERIAL	VALUE OF C	CONDUIT MATERIAL	VALUE OF C
Ductile pipe	100-140	Concrete	100-140
Cast iron	100-120	Asbestos cement	120-140
Galvanized steel below 50mm	55-120	Plastic pipes	120-140
Steel	100-140	Glass reinforced pipes	140-145

The **Colebrook-White** equation is used to iteratively solve for the Darcy-Weisbach friction factor (*f*) in the formula Head loss ($h_{f)}$ = *f* × L × v2/(2g × ID). It is the preferred equation for Full Flow (Closed Conduit) turbulent conditions, and the expression is:

$$\frac{1}{\sqrt{f}} = -2log(\frac{e}{14.8R} + \frac{2.51}{Re\sqrt{f}})$$

Where:

f = the friction factor is a function of: the roughness height, *e* (in m);

L = the pipeline length (in m),

ID = the actual internal diameter (in m);

R = the hydraulic radius (in m) = A/p (cross-sectional area/wetted perimeter);

v = the velocity (in m/s);

Re = Reynolds number (unit less), the ratio of inertial forces to viscous forces.

The head loss is best calculated using a spreadsheet incorporating a number of equations and for all practical purposes five iterations usually give sufficient accuracy. Required are the basic information about the pipeline, its actual internal diameter (ID), thickness (t) and type of any lining, the length (l), the flow (l/s), the kinematic viscosity (ν), and the roughness (k_s).

Viscosity is a measure of the resistance of a fluid which is being deformed by either shear stress or extensional stress. In general terms it is the resistance of a liquid to flow, or its "thickness" and may be thought of as a measure of fluid friction. The ratio of the viscous force to the inertial force, the latter characterised by the fluid density ρ, is termed the kinematic viscosity (ν), and defined as follows:

$$\nu = \mu / \rho$$

Where,

μ is the (dynamic or absolute) viscosity (in centipoise cP),

ρ is the density (in grams/cm^3),

ν is the kinematic viscosity (in centistokes cSt).

Kinematic viscosity (ν) is dependent on the temperature and within the usual range of water supply operation may be taken as follows:

TABLE 4.31: KINEMATIC VISCOSITY OF WATER AT DIFFERENT TEMPERATURES

Temperature (t) (°C)	Kinematic Viscosity (ν) (m^2/s) x 10^{-6}
10	1.307
20	1.004
30	0.801
40	0.658

The roughness, usually represented by (k or k_s) in mm is a linear measure of surface roughness taken as the diameter of uniformly graded sand particles that if so applied to the inside wall of a pipe would give the same resistance under rough turbulent flow. It is recommended that for design purposes, values for roughness after allowance for aging be adopted as follows:

TABLE 4.32: RECOMMENDED VALUES OF DIFFERENT PIPE ROUGHNESS'S (k_S)

Material and type of pipe	Condition of pipe	k_s in mm
Rigid PVCu		0.005
Polyethylene		0.003 to 0.015
new (ductile) cast iron	bitumen coated	0.10 to 0.15
	not bitumen coated	0.25 to 0.15
	cement mortar lined (factory/in-situ)	0.025 / 0.100
used (ductile) cast iron	evenly corroded	1.0 to 1.5
	slightly to heavily encrusted	1.5 to 3
	cleaned after several years in service	1.5
new seamless steel	rolled or drawn	0.02 to 0.05
new welded steel		0.04 to 0.1
new coated steel	zinc plated	0.10 to 0.15
	bitumen coated	0.050
	cement mortar lined (factory/in-situ)	0.025 / 0.100
	epoxy lined	0.010
	galvanised	0.01 to 0.10
used steel	evenly corroded	0.150
	slightly encrusted	0.15 to 0.4
	medium encrusted	1.5
	heavily encrusted	2 to 4

A calculation spreadsheet is as follows:

cells	B	C	D	E	F
2	**VELOCITY CALCULATION FOR: -**				
3	Cement-mortar lined steel pipe				
4	SELECTED	FROM STEEL PIPE DATA			
5	Nominal Diameter	Outside	Wall thickness	Lining	
6		Diameter		cement-mortar	
7	(mm)	(mm)	(mm)	(mm)	
8	80	88.9	3.2	4	
9					
10	Internal Diameter	(m)	=(C8-(D8+E8)*2)/1000		
11	Area	(m²)	=PI()*(D10)^2/4		
12	Velocity	(m/s)	=$D$17/1000/D11		
13					
14	**FRICTION HEAD INPUTS**			**RESULT**	
15	Pipeline Diameter	(mm)	=B8	**Friction Head**	
16	Length	(m)	1000	**(m)**	
17	Flow	(l/s)	10	**=D29*D$16*D12^2/2/D$20/D10**	
18	Rougness (ks)	(mm)	0.1		
19	Kinematic viscosity, v	(m²/s)	0.00000114		
20	Acc'n due to gravity, g	(m²/s)	9.81		
21					
22	**FRICTION HEAD CALCULATION**				
23	Reynolds No	Re	=D12*D10/D$19		
24	SELECT START VALUE	l0	0.022		
25	Colebrook-White Iterations	l1	=(-2*LOG(2.51/$D23/SQRT($D$24)+$D$18/1000/3.71/$D10))^(-2)		
26		l2	=(-2*LOG(2.51/$D23/SQRT(D25)+$D$18/1000/3.71/$D10))^(-2)		
27		l3	=(-2*LOG(2.51/$D23/SQRT(D26)+$D$18/1000/3.71/$D10))^(-2)		
28		l4	=(-2*LOG(2.51/$D23/SQRT(D27)+$D$18/1000/3.71/$D10))^(-2)		
29		l5	=(-2*LOG(2.51/$D23/SQRT(D28)+$D$18/1000/3.71/$D10))^(-2)		

With the results as shown below: -

cells	B	C	D	E	F
2	**VELOCITY CALCULATION FOR: -**				
3	Cement-mortar lined steel pipe				
4	SELECTED	FROM STEEL PIPE DATA			
5	Nominal Diameter	Outside	Wall thickness	Lining	
6		Diameter		cement-mortar	
7	(mm)	(mm)	(mm)	(mm)	
8	80	88.9	3.2	4	
9					
10	Internal Diameter	(m)	0.0745		
11	Area	(m^2)	0.0044		
12	Velocity	(m/s)	2.29		
13					
14	**FRICTION HEAD INPUTS**			**RESULT**	
15	Pipeline Diameter	(mm)	80	**Friction Head**	
16	Length	(m)	1000	**(m)**	
17	Flow	(l/s)	10	**81.41**	
18	Rougness (ks)	(mm)	0.1		
19	Kinematic viscosity, v	(m^2/s)	0.00000114		
20	Acc'n due to gravity, g	(m^2/s)	9.81		
21					
22	**FRICTION HEAD CALCULATION**				
23	Reynolds No	Re	149,916		
24	SELECT START VALUE	l0	0.022		
25	Colebrook-White Iterations	l1	0.022631992		
26		l2	0.022612195		
27		l3	0.022612803		
28		l4	0.022612784		
29		l5	0.022612785		

4.13.2 Head Losses in Fittings

Allowance for the additional losses due to standard fittings can be made by adding the equivalent lengths given in the table on the following page to the actual length of the main.

However when quoting the length of the pipeline and when plotting the profile or longitudinal section of a pipeline, care should be taken not to mix the actual surveyed length with the total equivalent length as errors in stated length and in presentation will then result.

The equivalent lengths are given in the following table for flow through the branches of tees and angle branches relates to the diameter at the small end.

There is negligible resistance to flow when it is towards the smaller end of a taper.

TABLE 4.33: LOSSES IN FITTINGS AS EQUIVALENT LENGTHS OF MAIN

NOMINAL DIAMETER OF FITTING	EQUIVALENT LENGTH OF MAIN							
	BENDS				TEES & 45^0 ANGLE BRANCHES FLOW THROUGH MAIN	TEES- FLOW THROUGH BRANCH	45^0 ANGLE BRANCHES FLOW THROUGH BRANCH	TAPER FLOW FROM SMALL END
	90^0	45^0	$22^1/_2{}^0$	$11^1/_4{}^0$				
	m	m	m	m	m	m	m	m
80	2.5	1.0	0.5	-	1.0	3.0	2.0	0.5
100	3.5	3.5	1.5	0.5	1.5	3.5	2.5	0.5
150	5.0	2.0	1.0	0.5	2.5	5.5	4.0	1.0
200	7.0	3.0	1.5	0.5	3.0	7.0	5.5	1.0
250	8.5	3.5	2.0	1.0	4.0	9.0	7.0	1.5
300	10.0	4.0	2.0	1.0	5.0	11.0	8.0	2.0
350	12.0	5.0	2.5	1.0	5.5	12.5	9.5	2.0
400	14.0	5.5	3.0	1.0	6.5	14.5	11.0	2.5
450	15.5	6.5	3.0	1.5	7.0	16.0	12.0	2.5
500	17.0	7.0	3.5	1.5	8.0	18.0	13.5	3.0
600	21.0	8.5	4.0	2.0	9.5	21.5	16.0	3.5
700	24.0	10.0	5.0	2.0	11.0	25.0	19.0	4.0
800	27.0	11.0	5.5	2.5	13.0	29.0	21.5	5.0
900	30.5	12.5	6.5	2.5	14.5	32.5	24.5	5.5
1000	34.0	14.0	7.0	3.0	16.0	36.0	27.0	6.0
1100	37.0	15.5	7.5	3.5	17.5	39.5	29.5	6.5
1200	41.0	17.0	8.5	3.5	19.0	43.5	32.0	7.0
1400	47.5	19.4	10.0	4.0	22.5	50.5	38.0	8.5
1600	54.5	22.5	11.0	4.5	25.5	57.5	43.0	9.5

4.13.3 Hydraulic Testing of Pipelines

All pipelines should be subjected to site test pressure for the purpose of checking:

a. The mechanical soundness and leak tightness of the pipes and fittings
b. The leak tightness of the joints.
c. The soundness of any construction work, in particular that of the anchorages.
d. The quality of workmanship of the pipe layers and jointers.

The tests applied to pipelines are generally hydraulic but air tests may be applied for certain specific applications.

Subject to any limitations imposed by the pressure rating of the joint or flanges, the test pressure to be applied should be not less than the greater of the following:

i. The working pressure times 1.5,
ii. The maximum sustained operating pressure, or the maximum static pressure, plus 50%, or
iii. The sum of the maximum sustained operating pressure (or the maximum static pressure) and the maximum calculated surge pressure.

The above to be subject to the recommended maximum hydraulic test pressure given in the specification or by the manufacturer. It should be established before a test commences that all anchorages are in position, that concrete anchorages have developed the required strength, and that where the joints of pipes are deflected to produce large radius curves, the backfill between the pipe body and the trench side met the required compaction density.

Air valves or suitable tapings need to be located at appropriate high points of the main to allow the air to escape whilst the main is being filled. Except in exceptional circumstances, pipelines should be laid to a minimum gradient. However if the pipeline is on a level grade, it may be necessary to bleed air off at several points to ensure complete evacuation. After the air has been evacuated all vent holes must be plugged.

Pipelines may be tested in one length preferably not exceeding 500 metres or in sections, the length of the sections being dependent upon the availability of water for testing, the difference in elevation between different parts of the pipeline, etc.

Where joints are left uncovered until testing has been completed, sufficient backfill material should be placed over the body of each pipe to prevent movement. However, joints in thermoplastic pipes should be covered during the progressive backfilling process but some temporary marking provided so that they can be relocated if necessary.

The simplicity and reliability of modern flexible joints has led to the practice of completing backfilling and reinstatement before the test is applied. This practice has the advantage of minimizing the disruption and inconvenience caused by lengths of open- trench.

After filling, the pipeline should be pressurized to the specified operating pressure and left for a period of time to achieve stable conditions.

The length of this period of time will depend on many factors such as slight movement of the pipeline under pressure, whether air is trapped in the pipeline or whether the pipeline has a concrete lining which absorbs water.

The pipeline is then pressurised up to the full test pressure and the section under test completely closed off. The test should be maintained for a period of time sufficient to reveal any defects in the pipes joints or anchorages. The period of time should not be less than one hour.

The test pressure should be measured at the lowest point of the section under test or alternatively an allowance should be made for the static head between the lowest point and the point of measurement, to ensure that the required test pressure is not exceeded at the lowest point.

If the pressure has dropped at: the end of the test period, the quantity of water required to bring the pressure up to full test pressure should be established, and for normal installations the test can be considered satisfactory if this quantity does not exceed 0.1 litres per millimetre of pipe diameter per kilometre of pipeline per 24 hours at working pressure. With the exception of testing non-pressure mains at very low pressures, air testing is to be avoided if possible due to the hazards inherent in containing large volumes of compressed air. Experience has shown that, to prove the leak tightness of joints, the pressure applied when testing pressure water mains need not exceed 2 Bar.

4.14 PIPES AND PIPELINES

4.14.1 General Requirements

There are five principle requirements for a pipeline, namely: -

a. It must convey the quantity of water required
b. It must be capable of resisting all external and internal forces
c. It must be durable and meet the design working life
d. It must be properly laid and embedded
e. The material from which it is made should not adversely affect the quality of the water being conveyed.

The pipeline must be designed to withstand the following:

i. Internal test pressure of water
ii. Water hammer (positive surge)
iii. Vacuum and negative surge
iv. External pressures when laid below ground (overburden and surcharge)
v. Conveyance water temperature (thermoplastic pipes)
vi. Maximum working temperature (ferrous pipe coatings)
vii. Temperature stresses when laid above ground
viii. Flexural stresses when laid over supports, constructed at intervals or on bridges
ix. Longitudinal stresses due to flow at tees, tapers and bends
x. Foundation reaction depending upon the nature of support
xi. Handling stresses

For flexible pipes (thermoplastic and steel) the following criteria must be met:

- The pipe deflection (out-or-roundness) must not exceed the allowable limit;
- The combined stress or stain in the pipe wall must not exceed the allowable limit, and
- The factor of safety against buckling must be adequate.

For semi-rigid pipes (ductile iron) the following criteria must be met:

- The pipe deflection (out-or-roundness) must not exceed the allowable limit, and
- The pipe wall bending stress must not exceed the allowable limit.

4.14.2 Requirements for Pipeline Design

4.14.2.1 Basis of Design

The basis for any design is the knowledge and past experience of others that has resulted in the development and publishing of **National and International Standards**. Such Standards govern the world of manufactured goods and services. As a result, it has become common practice in recent years, to specify that goods and services must meet the requirements of international ISO standards.

However, because such standards require the approval of a large number of countries affiliated to the Organisation, and where they involve the manufacture of goods, they may provide a minimum basic requirement only. In such instances it is considered desirable to consider local circumstances and to strengthen requirements by reference to more rigorous National Standards and local knowledge.

In the water supply sector, national standards that may do this include TBS (Tanzania), BS (UK), DIN (Germany), AWWA (USA) and SABS (South Africa). Also, there is now the European Regional Standard, the EuroNorm (EN), although even here the resulting standard may in part be less rigorous than in some of the national standards it replaces. When referring to such standards, climatic and other differences must however be taken into account. For example, effects of freezing temperatures and roof snow-loads are clearly irrelevant in a Tanzanian context but the risk of third party damage especially due to vandalism, impact loads from over-loaded vehicles and the effects of high temperatures are very important.

Even within National Standards themselves, the influence of a powerful manufacturing lobby may on occasions be discernable. It is therefore in the interest of Clients for their Designers to familiarise themselves with what might be termed, *'the fine print'* in such Standards when drawing up or agreeing to their use in Contract Specifications.

Codes of Practice, Manuals and Industry Specifications are available to guide Designers and Clients and these come under a variety of titles. Again here, climatic and other differences between the countries of origin and Tanzania must be carefully considered. Amongst such documents those referred to here and from which the various tables have been abstracted are indicated below unless specifically indicated otherwise within the table itself[< >].

4.1.1.3 *Basic Data Required*

Before any pipeline design can take place it is necessary to assemble all the basic data that is required. From the hydraulic requirements, the most probable internal diameter or diameters will have been established. To this must be added knowledge of the location and hence of the native soil properties, the design pressures, the overburden pressure and the possible surcharge pressures. Knowledge about the behaviour and design requirements of the alternative pipes under consideration is also necessary.

Only when these are all known can structural design of the pipeline begin.

4.1.1.3 *Pipe Trenches, Soil and Surrounds*

Common to all pipelines laid below ground is the pipe trench and because of variations in definition it has been considered necessary to provide a clear and precise picture that is used throughout this Manual. This is done by means of a schematic or diagram on the following page.

Soil and Surrounds play a key role in pipeline design. Knowledge of the natural or 'native' soils through which a pipe is to be laid is often the key to success or failure. The strength of the native soil within which the pipe trench is excavated (or the embankment material placed) is important for the design of ALL pipelines.

[< >] EN 1295:1998 Structural Design of Buried Pipelines under Various Conditions of Loading - Part 1: General Requirements — BSI, 1998

Pipe Materials Selection Manual, Water Supply, (complies with EN 1295) — 2nd Edition, WRc, June 1995

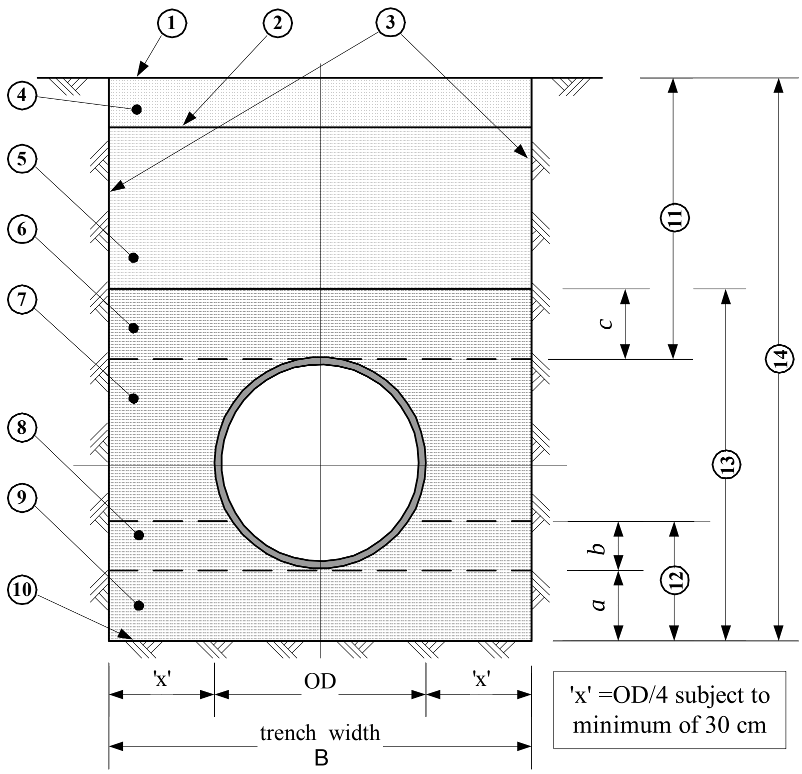
uPVC Pipe Selection Manual, — WRc, 2002

Steel Pipe – A Guide for Design and Installation (Manual of Water Supply Practices) M11, — 4th Edition AWWA, 2004

CP BS 2010-2, & BS 8010, Pipelines, Design and Construction of Steel Pipelines in Land — BSI 1970

CP 312-1 to 3 Codes of Practice for plastics pipework (thermoplastics material). — BSI 1973

If the native soil is less stiff than the embedment material that is placed in the trench around the pipe, this will reduce the support given to the sides of flexible pipes whilst semi-rigid pipes may experience a loss of both horizontal and vertical support. Soil stiffness is quantified by the modulus of soil reaction (E'), for the soil surround by E'_2, and for the native soil by E'_3.



1 Surface
2 Bottom of road or railway construction (if any)
3 Trench walls
4 Top soil
5 Main backfill
6 Initial backfill
7 Sidefill
8 Upper bedding
9 Lower bedding
10 Trench bottom
11 Depth of cover
12 Depth of bedding
13 Depth of embedment
14 Trench depth
a *Depth of lower bedding*
b *Depth of upper bedding*
c *Depth of initial backfill*

Note 1: compaction is the deliberate densification of soil during the construction process, whereas consolidation is the subsequent time-dependant densification after construction. Attaining cited minimum compaction requirements is at the supervisors disgression.

Figure: Trench Installation

Note 2: the embedment is the arrangement and type(s) of material(s) around a buried pipeline which contribute to its structural performance. Attaining cited minimum compaction requirements is mandatory for embedment.

Tables 4.34 to 4.40 have been abstracted from the WRC Pipe Materials Selection Manual 1995 (BSEN 1295 part 1:B.1.1.2).

A guide to soil modulus values for various soil types is given below: -

TABLE: 4.34: GUIDE TO SOIL MODULUS VALUES IN VARIOUS CONDITIONS

SOIL TYPE	SOIL MODULUS (E'_3) IN (MN/m^3)				
	VERY DENSE	DENSE	MEDIUM DENSE	LOOSE	VERY LOOSE
Gravel	Over 40	15-40	9-15	5-9	3-5
Sand	15-20	9-15	4-9	2-4	1-2
Clayey silty sand	10-15	6-10	2.5-6	1.5-2.5	0.5-1.5
Clay	Very hard	11-14			
	Hard	10–11		Firm	3-4
	Very Stiff	6-10		Soft	1.5-3
	Stiff	4-6		Very Soft	0-1.5
The values quoted indicate likely values at shallow depths and with groundwater present. They will therefore be conservative for pipelines above groundwater level and at depths greater than one metre. For trenches cut in rock a value of 40 MN/m^3can be taken as a conservative figure.					

Pipelines derive their strength from a combination of the intrinsic strength of the pipe and from the support obtained from the embedment, which as noted above, in turn derives some of its strength from the surrounding native soil. If the embedment is inappropriate because of its content or because it is insufficiently compacted then the expected working life of the pipeline is unlikely to be achieved. All too often, poor performance is blamed on the pipe manufacturer when in fact it is the installation which was at fault. This is particularly true with PVCU and where performance of PVC pipes in particular has been judged satisfactory, proper attention to installation is inevitably a factor in this [< >].

That part of the embedment that supports the pipe is termed the bedding. The structural behaviour of all classes of pipe are influenced by the bedding support angle which is a measure of the angular extent of the arc extending upwards from the pipe invert which bears on firm material capable of distributing the reaction forces from the foundation. Bedding classification is related to this bedding support angle but for all practical purposes can be taken as 180 degrees with the limitations as shown on the following table: -

TABLE 4.35: BEDDING CLASSES

ANGULAR EXTENT OF BEDDING MATERIAL	BEDDING CLASSIFICATION FOR VARIOUS PIPE CLASSES		
	SEMI-RIGID (D.I.)	FLEXIBLE (STEEL)	FLEXIBLE (THERMOPLASTIC)
360^0	Classes S1 to S4	Classes S1 to S3	Classes S1 to S2
Note: The angular extent of bedding is not to be confused with the bedding support angle			

The Bedding Classes and Embedment Materials to be used are as follows: -

TABLE 4.36: BEDDING FOR THERMOPLASTIC PIPES

EMBEDMENT CLASS	LOWER AND UPPER BEDDING MATERIAL ALLOWED	NOTES
S1 and S2	Class S1: Gravel – single size Class S2: Gravel – graded	processed granular materials required for all thermoplastic pipes

TABLE 4.37: BEDDING MATERIAL FOR THERMOPLASTIC PIPES

NOMINAL PIPE DIAMETER (mm)	GRADING (BASED ON ASTM) [ISO SIEVE SIZE IN mm]	
	S1 BEDDING	S2 BEDDING
80	2.0 mm single-size gravel	2.0 mm single-size gravel
100	2.0 mm single-size gravel	2.0 mm single-size gravel
150	2.0 or 1.4 mm single-size gravel	1.4 to 4.0 mm graded
200 to 500	2.0, 1.4 or 0.85 single-size gravel	1.4 to 4.0 mm graded or 0.85 to 4.0 mm graded
> 500	2.0, 1.4, 0.85 mm single-size crushed rock, or gravel	1.3 to 4.0 mm graded or 0.85 to 4.0 mm graded
The ISO Sieve Sizes are the closest in mm to the ASTM sizes in fractions of an inch. These gradings will also apply to ferrous pipes in certain circumstances (see below)		

[< >] Minimizing Failures to PVC Water Mains, Roy Brander, P.Eng., Senior Eng., Waterworks, City of Calgary, 2004

For the sidefill and initial backfill embedment layers for thermoplastic pipes the requirements are given in Table 4.36.

For flexible and semi-rigid ferrous pipes the following Table indicates requirements:

TABLE 4.38: EMBEDMENT FOR FLEXIBLE AND SEMI RIGID FERROUS PIPES

EMBEDMENT CLASS	EMBEDMENT MATERIAL ALLOWED	NOTES
S1 and S2	Class S1: Gravel – single size Class S2: Gravel – graded	Normally processed granular materials except where otherwise specified as bedding for ferrous pipes
S3 and S4	Class S3: Sand and coarse grained soil with less than 12% fines Class S4: Coarse grained soil with more than 12% fines OR Fine grained soil, liquid limit less then 50%, medium to no plasticity and more than 25% coarse grained material	These represent "as dug" soils but require particularly close control when used with low stiffness pipes. Class S3 shall be used for epoxy lined steel pipes whilst class S4 is suitable for cement mortar lined ferrous pipes
In areas prone to waterlogging or where specifically called for, the S1 or S2 material to be used for lower and upper bedding shall be as given in Table 4.34 for Thermoplastic pipes.		

Embedment properties can be obtained from the following Table:

TABLE 4.39: EMBEDMENT PROPERTIES FOR ALL FLEXIBLE AND SEMI-RIGID PIPES

EMBED-MENT CLASS & DEFLECT'N COEFF.	COMP-ACTION % MPD	MODULUS OF SOIL REACTION E'_2 MN/m^2	DEFLECT-ION LAG FACTOR D_L	STRAIN FACTORS (D_f) FOR VARIOUS PIPE STIFFNESS IN kN/m^2 (Req'd for Thermoplastic Pipes only)					
				1.25	2.5	5.0	10	15	≥ 30
Class S1 k_x = 0.083	Uncompacted 80 85 90	5 7 7 10	1.50 1.25 1.00 1.00	4.70 4.70 4.70 4.70	4.50 4.50 4.50 4.50	4.30 4.30 4.30 4.30	4.00 4.00 4.00 4.00	3.75 3.75 3.75 3.75	3.00 3.00 3.25 3.50
Class S2 k_x = 0.083	Uncompacted 80 85 90	3 5 7 10	1.50 1.25 1.00 1.00	4.70 4.70 4.70 4.70	4.50 4.50 4.50 4.50	4.30 4.30 4.30 4.30	4.00 4.00 4.00 4.00	3.75 3.75 3.75 3.75	3.00 3.00 3.25 3.50
Class S3 k_x = 0.100	80 85 90	3 5 7	2.00 1.50 1.25	3.50 6.20 7.75	3.40 5.50 6.60	3.20 4.76 5.50	3.10 4.25 4.70	3.00 4.00 4.25	3.00 3.25 3.50
Class S4 k_x = 0.100	85 90	3 5	1.50 1.25	6.20 7.75	5.50 6.60	4.75 5.50	4.25 4.70	4.00 4.25	3.50 3.50

Notes:
1. Pipe stiffness referred to in this Table are initial values
2. Where the Designer can be certain that initial pressurisation within one year of backfilling, a value of 1.0 may be taken for the Deflection Lag Factor
3. For semi-rigid pipes EN 1295 permits Density (MDP) values of 95% although achieving this in practice is difficult.
4. Uncompacted material for S1 and S2 embedment **MUST** only ever be used where the material is strictly gravel and sized/graded as indicated. Where in doubt then it **MUST** be compacted.

The Modified Proctor Density (MDP) test first determines the maximum density achievable for the materials and uses this figure as a reference. It then tests the effects of moisture on soil density. The soil reference value is expressed as a percentage of density. A small soil sample is taken from the embedment layer. A standard 10 lb weight is dropped 25 times on each soil layer through a distance of 15”. The material is then weighed and oven dried for 12 hours in order to evaluate water content.

4.14.2.4 Overburden Pressure and Surface Surcharge Loadings

Overburden pressure, p_e, is the product of the depth of cover, H, and the unit weight of the overburden, γ_s, such that $p_e = H \times \gamma_s$.

Surface surcharge loadings are calculated from vehicle wheel loads and impact factors intended to represent the conditions likely to be encountered in various locations. The figures apply to smooth surfaced roads so need uprating for normal tarmaced roads and even more so for dirt roads and roads with damaged roads surfaces and for vehicles travelling along unpaved road verges. The values given are also for vehicles complying with weight limitations and speed limits so that adjustment factors are required to allow for impacts on roads and verges and for overloaded vehicles. Because data on these effects is limited[< >] it is suggested that impact factors of at least 1.50 be applied for reasonable roads surfaces[< >] , and 1.67 for dirt and damaged roads and for road verges whilst 1.20 be applied to allow for overloading.

TABLE 4.40: VERTICAL PRESSURE ON PIPES DUE TO TRAFFIC LOADING (IMPACT FACTOR = 1)

DEPTH OF COVER (m)	VERTICAL PRESSURE ON PIPE DUE TO TRAFFIC (p_s)		
	MAJOR ROADS (kN/m^2)	NORMAL ROADS (kN/m^2)	OFF-ROAD (kN/m^2)
0.6	-	96	50
0.7	-	74	39
0.8	-	59	32
0.9	55	48	27
1.0	49	41	23
1.2	41	30	17
1.4	35	23	14
1.6	30	18	11
1.8	27	15	9
2.0	24	13	8
2.5	19	9	6
3.0	16	6	4
3.5	13	5	3
4.0	12	4	3
5.0	9	3	2
6.0	8	2	1

“Major roads” are those on which multi-wheeled trailers can be expected. The “normal road” loadings are based on 7 tonne wheel loads. Adjustment upwards for impact and vehicle overload is required. Pipes laid adjacent to roads should be designed to accept the full loading for that class of road inclusive of impact and overload multipliers.

< > “The dynamic weight of a truck bouncing along a rough road can be as much as twice its static weight.” Evaluating Pavement Surfaces: LISA and RQI MDOT’s Lightweight Inertial Surface Analyzer (LISA) & the Michigan Ride Quality Index (RQI). Materials and Technology Research Record, Issue no 79, June’96.

< > WRc Pipe Materials Selection Manual Section 3.2.3.2, page 38.

4.14.3 Pipes and Pipe Wall Materials

4.14.3.1 General Considerations

In the following sections, first a qualitative assessment and then quantitative evaluations, laying down design procedures, are made for the four more common pipe alternatives used in Tanzania, namely unplasticised Poly Vinyl Chloride (uPVC or PVCu), High Density Polyethylene, PE_{100} (PE_{HD}), and Zinc (GS) and Barrier Coated < > Steel and Ductile Iron (D.I.).

Because of their relatively short working life, GS pipes should only be considered for outside use in external diameters (DE) of DN 65 <> and less, but limited to non-corrosive soils, rocky and other difficult areas away from the coastal belt. Otherwise, additional protection must be applied and organic coatings such as bituminous or epoxy paint in at least two coats are amongst the better ones. In addition joints, and any exposed threads, should be cleaned, prepared and wrapped in a suitable adhesive tape.

PVCu, PE_{HD} and barrier coated Steel pipes are classified structurally as flexible pipes, or pipes whose properties are such that the first limit state reached is either excessive deformation or buckling collapse. As a result, and when designing pipelines for such materials, both these factors should be checked.

Ductile Iron (D.I.) is classified structurally as a semi-rigid pipe being a pipe that can deform enough to redistribute some of the overburden pressure to the side fill but is stiff enough to rule out the possibility of buckling. The first limit state reached may be either excessive deformation or excessive wall bending stress.

Towards the end of the Chapter, Life-Cycle Assessment for Environmental Costs and Values of these four pipe types is discussed to enable Designers to take account of this in line with the latest requirements of the National Environment Monitoring Council (NEMC).

4.14.3.2 Qualitative Assessment of More Common Pipe Types

Before attempting to evaluate each of the pipe wall materials and provide details for design, it is useful to consider these four pipe types qualitatively. The two thermoplastics pipes considered, namely PVCu and PE_{HD} have a number of things in common as well as significant differences.

The other two alternatives of steel and D.I. are both ferrous and being adjacent to each other in the electro-chemical series are prone to corrosion at approximately the same rate < >. They must therefore be appropriately coated and lined and again have both similarities and differences.

All but D.I. pipes and fittings and fittings for steel pipes are manufactured in Tanzania although such fittings are manufactured in Nairobi, Kenya.

Design requirements, cost and diameter range are not considered here. However, acceptable standards of quality control during manufacture and installation are assumed to be achievable.

The following table, partly derived from WRc Pipe Materials Selection Manual, 1995 considers the principle advantages and disadvantages of each of the four pipewall materials: -

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< > Barrier coated steel pipes, referred to hereafter as Steel pipes have an electrically resistive coating as opposed to being hot dipped in zinc, the latter usually being incorrectly referred to as being 'galvanised'. 'Galvanised' steel pipes are variously referred to as GS or GI pipes, but the former is to be preferred.

< > As at Dec. 2006, the smallest diameter of barrier coated steel pipe available locally was DN80.

< > Strictly speaking for D. I. it is graphitisation a form of corrosion which occurs.

TABLE 4.41: ADVANTAGES AND DISADVANTAGES OF THE FOUR PIPEWALL MATERIALS

PVCU PIPE ADVANTAGES	PVCU PIPE DISADVANTAGES
• Relatively low energy requirements when considering all manufacturing stages. • Light in weight and relatively easy to lay. • Fairly easy to work on during subsequent repairs and when making new connections to the pipe. • Low supply cost.	• Environmentally unfriendly in raw material manufacture & in eventual disposal. (risk of release of dioxins) and also in disposal if lead or organotins used as a stabiliser. • Requires accurate mixing of several raw materials during manufacture requiring high standard of quality control. • Length restricted to 6m. • Prone to vandalism & illegal connection. As a result has a high to very high risk of water loss as a % of the total flow if risks present. • Needs to be de-rated for temperatures above 25 ^{0}C and should not be stored at temperatures higher than this. Storage stacks need to be limited to 1m height. • Cannot withstand ultra violet sun rays and deteriorates upon exposure with accompanying risk of migration of chemical constituents. • Requires to be laid in relatively deep trenches. • Requires a graded granular material (fine gravel) bed and surround with compaction to at least 80 and preferably 90% MPD. • Requires that account be taken of either the short term Pressure Fluctuation when considering the combined stress OR the long term Hydrodynamic Re-rating factor when determining the Maximum Permissible Design Stress. This is often overlooked leading to inadequate wall thickness.. • In unpaved road areas in particular, at risk from overloaded vehicular traffic, such that wall thickness has to be increased to counterbalance this. • In situations where there is a risk of vacuum conditions being created during pipeline filling or draining and during water rationing within a distribution system, wall thickness has to be increased. • Requires use of metallic strip laid in bedding above pipe for ease of subsequent location. • Requires separate corrosion resistant saddles for consumer connections. • Local evidence to date suggests can have a limited working life esp. if inadequately designed for and/or poorly laid.
PE_{HD} PIPE ADVANTAGES	**PE_{HD} PIPE DISADVANTAGES**
• Straightforward to manufacture and hence achieve consistently good quality. • Moderately low energy costs when considering all manufacturing stages. • Easy to lay • Has a low to very low risk of water loss as a % of the total flow except where there is risk of vandalism. • Fairly easy to work on during subsequent repairs and connections to the pipe. • Available evidence suggests can be expected to have a long working life	• Prone to vandalism and illegal connection. • Requires to be laid in relatively deep trenches. • Needs a graded granular material (fine gravel) bed & surround with thorough compaction to at least 80% MPD. • In unpaved road areas at risk from overloaded vehicular traffic, wall thickness has to be increased. • Requires use of metallic strip laid in bedding above pipe for ease of subsequent location. • In situations where there is a risk of vacuum conditions being created during pipeline filling or draining and during water rationing within the distribution system, wall thickness have to be increased. • Requires relatively expensive consumer connection saddles, although these are superior to those needed for other pipes.

Cont'd

BARRIER COATED STEEL PIPE ADVANTAGES	STEEL PIPE DISADVANTAGES
• Available in lengths of up to 12 m. • Relatively free from risk of vandalism and illegal connection in urban environments. • Has no temperature related limitations. The limit to the height of stacks during storage and handling is only related to what is practicable. • Mains location straightforward. • Leal detection methods well developed. • Can be laid in relatively shallow depth trenches. • For cement-mortar lined pipe, does not specifically require a graded granular material (gravel) embedment and thorough compaction whilst desirable is not essential. • Suitable for use in areas subjected to frequent heavy vehicular traffic loading. • Suitable for vacuum conditions created during pipeline filling or draining especially during water rationing within the distribution system. • For smaller diameters and with combined pipe coupler-ferrules provides an easy and cheap way for connecting of consumers. • Appropriately coated, lined and laid & jointed can be expected to have a working life of at least 40 years.	• High to very high energy costs when considering all stages of manufacture. • Without suitable coating and lining will deteriorate due to corrosion. • Traditional coatings especially liable to damage when handling prior to installation. • All but smallest diameters requires mechanical lifting equipment • Moderately difficult to manipulate the pipe during subsequent repairs and insert of fittings. • Has a moderate to high risk of water loss as a % of the total flow. • For epoxy lined pipes, care is required in both selection of embedment material and in compaction to minimise diametrical deflection.
DUCTILE IRON PIPE ADVANTAGES	**DUCTILE IRON PIPE DISADVANTAGES**
• Least prone to risk of vandalism and illegal connection. • Has a moderate to low risk of water loss as a % of the total flow. • Has no temperature related limitations. The limit to the height of stacks during storage and handling is only related to what is practicable. • Can withstand ultra violet sun rays during extended exposure. • Can be laid in relatively shallow depth trenches. • Does not specifically require a graded granular material (gravel) embedment and thorough compaction whilst desirable is not essential. • Suitable for use in areas subjected to frequent heavy vehicular traffic loading. • Suitable for vacuum conditions created during pipeline filling or draining especially during water rationing within the distribution system. • Appropriately coated, lined and laid & jointed has a working life of at least 40 years.	• High energy costs when considering all stages of manufacture. • Available only in 6- 8m lengths. • Requires mechanical lifting equipment. • Requires separate corrosion resistant saddles for consumer connections. • Without suitable coating and lining will deteriorate due to corrosion (graphitisation). • Traditional coatings are especially liable to damage when handling prior to installation. • Loose wrap polythene coating should not be used in urban areas as liable to damage when other services laid nearby. • Difficult to work on the pipe during subsequent repairs and insert of fittings. • Requires importation of bulky and heavy completed goods.

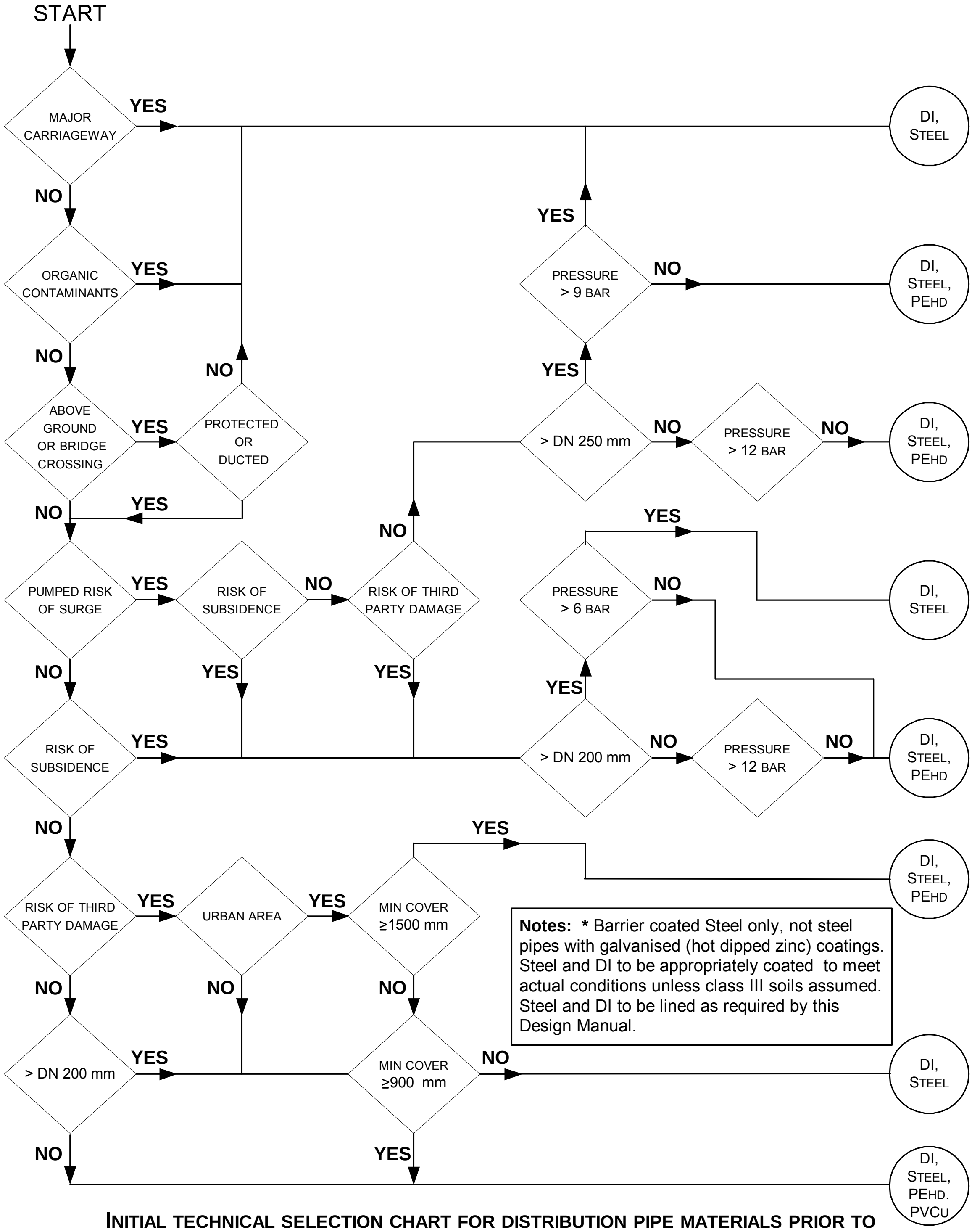
4.14.3.3 Initial Technical Selection for Zero Failures Approach

A zero-failures approach should only be adopted for rural areas and urban secondary and tertiary distribution systems with diameters not exceeding DE250 mm.

For larger diameters and trunk mains, a total cost of ownership approach should be the minimum design level adopted. It is however recommended that whenever time and cost allows, such an approach should also be followed for all urban networks.

The first step in a zero-failures approach is to consider an initial selection based on the technical selection chart on the following page. This will then either limit the alternative types of pipe to be considered or impose certain minimum criteria on their selection.

Subsequent consideration and design can then be limited to those types of pipe that pass the initial selection criteria. The flow chart (WRc 1995) presented below is for such an initial selection.



INITIAL TECHNICAL SELECTION CHART FOR DISTRIBUTION PIPE MATERIALS PRIOR TO DESIGN WHEN A ZERO-FAILURES APPROACH IS BEING ADOPTED.
PIPE MATERIALS CONSIDERED ARE PVCU, PEHD, STEEL* AND DI

4.14.3.4 Thermoplastic Pipes

Thermoplastic pipes are made from ethylene obtained from oil (PE) and ethylene and chlorine (PVCu). Such pipes are produced by extrusion and extrusion temperature is critical to ensure the end product is satisfactory.

This can mean that power fluctuations or other problems during production result in an unacceptable quality and then such pipes are ground up to be reused as rework material. However, for pressure applications, the use of rework material should be forbidden.

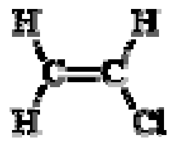
For local plastic pipe production in particular, it is important to be satisfied that the imported materials that make up the compound are of suitable quality as it is not unknown for unscrupulous raw material manufacturers to 'dump' below standard or life-expired products onto overseas markets. At the project implementation stage, raw material certification should therefore be insisted upon.

In addition, pipe manufacturers should be required to have acceptable quality assurance procedures both in place and adhered to and purchasers should have the right to inspect without prior notice and/or to station a third-party inspector of their choice at the factory during production runs. Amongst other things, such a third party inspector should ensure that all the various chemical and other tests required are in fact undertaken and satisfactory results obtained. The inspector should also assure himself that the storage requirements of the pipes at the factory prior to delivery are followed.

As viscous elastic materials the breaking strength of thermoplastic pipes is both age and temperature dependant and working pressures and decreases with time so that they have to be de-rated to allow for this. The ageing process is speeded up by excessive loads and surge. However and except where organic contaminants are present in the soil, thermoplastic pipes have no corrosion problems, being immune to galvanic and electrolytic attack for all natural ground conditions as well as catering for the quality of water being conveyed without problem.

a) PVCu (unplasticised Poly Vinyl Chloride) and PVCu Pipes

The main raw materials for PVC are ethane or ethylene produced when refining oil and chlorine produced from salt. These are combined to form vinyl chloride, a monomer gas with chlorine making up about 57% by weight. This is then polymerised to form polyvinyl chloride, a fine white powder or resin. Chemically, vinyl chloride is C_2H_3Cl, the chemical structure being as shown: -



Additives are an essential component of PVC pipes. Some of these additives are stabilisers including heavy metals such as lead or organotin or alternatively inorganic compounds, whilst others give it colour.

Stabilisers allow the PVC to be processed without degradation during extrusion, provide strength and durability, and minimise migration of chemicals from the PVC. Pigments are used selectively to provide colour. Fungicides are also added to stop fungi from eating the other additives.

Like all PVC products, PVCu pipe gets its properties from combinations of some of these additives and modifiers which are mixed with PVC resin. These form the raw materials that are procured by PVC pipe manufacturers for their pipe production. Once the additives and modifiers have been combined with the resin, the resulting material is called a PVC compound, and is in granular form.

These granules are then melted down, blended thoroughly and extruded into pipe. Providing this melting and blending process is done properly, the finished pipe will be strong and safe in use provided there is full compliance with the criteria of nationally and internationally accepted physical property and general performance standards and in the requirements regarding storage temperature and protection from UV light before installation and use.

Advantages of PVCu pipes include:

- Cheapness
- Good acid and alkali resistance.
- Flame-retardant.
- Stiff and strong.
- Good vapour barrier properties.
- Reasonable UV resistance.

Disadvantages include:

- Difficult to melt process, requires modification.
- Limited solvent stress cracking resistance.
- Becomes brittle at (c 5°C (40°F)) unless impact modified.
- Has a relatively low continuous service temperature of 50°C (120°F).

Regrettably, PVC cannot be regarded as environmentally friendly when considered across the whole cycle from manufacture to disposal and this is discussed further in Section 4.20 under Life Cycle Assessment.

Hereinafter, the continuous operating pressure exclusive of surge as PFA.

PVCU pressure pipes generally come in nominal 6 metre lengths and although manufactured in diameters ranging from DE50 to DE600, they are normally found to be economic in diameters of DE225 and less. Originally manufactured in pressure classes starting at PFA 3 Bar, they now only manufactured from 6 Bar upwards. However and when design considerations additional to those related to hydraulic pressure are taken into consideration, the wall thickness of a 6 Bar pipe will rarely be sufficient so that in reality only pressure classes from 8 Bar upwards up to DE 90 and 10 Bar upwards for larger diameters need to be considered.

PVCu pipes are light and easy to handle but need care in both handling and storage. This often has negative consequences resulting in carelessness in use and poor installation practices.

They must be protected against impact loads, stored at temperatures of 25^{0}C or less and never left out in the sun. Ultra violet light changes the molecular composition, and where heavy metals are used as stabilisers brings the risk of their migration to the inner and outer surfaces and hence into the water being conveyed especially at elevated temperatures [<>]. Heat distorts the shape of the pipe.

Due to different design approaches that can be adopted, allowance should either be made for the greater of the short term Pressure Fluctuation in checking for the combined stress OR the long term Hydrodynamic Re-rating factor in the Maximum Permissible Design Stress but not both.

[<>] Migration of lead from unplasticized polyvinyl chloride pipes.
Muhammad H. Al-Malack, KFUPM Research Institute, Journal of Hazardous Materials, Dec'2000.

Although manufactured with both socket and spigot rubber ring joints and in the smaller diameters for solvent welded jointing, only rubber ring joints should be used as these can take up slight pipe ovality whereas solvent cement cannot.

Whilst there is now standardisation in nominal or outside diameters, this was not always the case as prior to 1968, PVC pipes produced to British Standards were to imperial dimensions. Hence and where tying works between old and new pipes are involved, Designers are warned to take note of this and where possible use joints that can accommodate such differences.

PVCu pipes are manufactured in a fairly limited range of sizes and pressure classes. For PVCu pipes to either ISO 4422-2:1996 or to EN 1452-2:1999, in the diameter range from DE20 to DE90 there are 7 nominal diameters, realistically 5 pressure classes (PFA 8, PFA10, PFA 12.5, PFA 16 and PFA 20) giving 35 possible combinations. Between DE110 and DE315, there are 10 nominal diameters, realistically 5 pressure classes (PFA 10, PFA 12.5, PFA 16, PFA 20 and PFA 25) giving 46 possible combinations (PFA 25 diameter range limited to DE110 to DE200). Wall thickness range between 1.5 mm (DE20, PFA 16) and 23.2 mm (DE315, PFA 20). Because of the extrusion process, tolerances are fine and average internal diameters can be used for flow calculations.

PVCu pipes are sometimes referred to by Pipe Series (S) or by a Standard Dimension Ratio (SDR). The pipe series is a dimensionless number related to the geometry of the pipe, its outside diameter, DE, and its nominal wall thickness, s. The standard dimension ratio approximates to the ratio of the outside diameter of a pipe wall to its thickness, such that:

$$S = (DE - s) / 2s \qquad \text{and,} \qquad SDR = 2S + 1$$

This can be confusing for those more familiar with nominal pressure ratings so that a comparison is given in the following table together with the nominal wall thickness.

TABLE 4.42: PVCU PIPE SERIES, PRESSURE CLASSES AND WALL THICKNESS

DE	NOMINAL WALL THICKNESS, PIPE SERIES S AND SDR				
	S 12.5 (SDR 26)	S 10 (SDR 21)	S 8 (SDR 17)	S 6.3 (SDR 13.6)	S 5 (SDR 11)
	PFA 8	PFA 10	PFA 12.5	PFA 16	PFA 20
20	---	---	---	1.5	1.9
25	---	---	1.5	1.9	2.3
32	1.5	1.6	1.9	2.4	2.9
40	1.6	1.9	2.4	3.0	3.7
50	2.0	2.4	3.0	3.7	4.6
63	2.5	3.0	3.8	4.7	5.8
75	2.9	3.6	4.5	5.6	6.8
90	3.5	4.3	5.4	6.7	8.2
NOMINAL PRESSURE BASED ON DESIGN COEFFICIENT C = 2.0					
DE	PFA 10	PFA 12.5	PFA 16	PFA 20	PFA 25
110	4.2	5.3	6.6	8.1	10.0
125	4.8	6.0	7.4	9.2	11.4
140	5.4	6.7	8.3	10.3	12.7
160	6.2	7.7	9.5	11.8	14.6
180	6.9	8.6	10.7	13.3	16.4
200	7.7	9.6	11.9	14.7	18.2

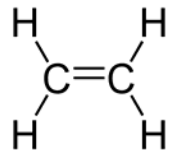
Cont'd

DE	PFA 10	PFA 12.5	PFA 16	PFA 20	PFA 25
225	8.6	10.8	13.4	16.6	---
250	9.6	11.9	14.8	18.4	---
280	10.7	13.4	16.6	20.6	---
315	12.1	15.0	18.7	23.2	---

For more detailed information, it is recommended that reference is made to ISO 4422, 1-2:1996: EN 1452, 1-5:2000: DIN 8061; and SABS 966-1:2000.

b) Polyethylene and Polyethylene Pipes

Polyethylene is one of a group of plastics known as polyolefins. The raw material for polyethylene is the monomer ethane or ethylene obtained when refining oil which is then used to create this polymer also known as polythene. The ethane molecule (known almost universally as ethylene), has the following chemical formula C_2H_4 or $CH_2{=}CH_2$ and is structured as shown: -



Various polymerization processes exist. Polyethylene can be produced through radical polymerization, anionic addition polymerization, ion coordination polymerization or cationic addition polymerization. This is because ethene does not have any substituent groups which influence the stability of the propagation head of the polymer. Each of these methods results in a different type of polyethylene.

Polyolefins are simple polymer structures that do not need plasticizers, although they do use additives such as UV and heat stabilizers, antioxidants and in some applications flame retardants. Polyolefins pose fewer risks than most other plastics and have the highest potential for mechanical recycling. Both PE and another polyolefin, polypropylene (PP) are versatile and cheap, and can be designed to replace almost all PVC applications. PE can be made either hard, or very flexible, without the use of plasticizers.

In comparison with PVC, polyolefins use fewer problematic additives, have reduced leaching potential in landfills, reduced potential for dioxin formation during burning (provided that brominated/chlorinated flame retardants are not used), and reduced technical problems and costs during recycling.

PE pipes are produced in low, medium and high densities but for pressure applications it is high density PE pipes (PEHD) using P_{100} resin that should be specified.

PE_{100} resin is a high molecular mass grade with good impact strength and high abrasion, chemical, and UV resistance, complying with the ISO 4427 requirements of PE_{100} pipe where transportation of potable water, is required. It can be extruded using conventional techniques at temperature profiles between 190^0 and 230°C with a melt temperature between 200^0 and 220°C.

PE is much less of a long term environmental problem than PVC and the manufacturing process for PE pipe is simpler and easier to control and as a result is more consistently able to attain a high quality.

Other advantages of PE pipes include: -

- Its light weight
- Ease of installation
- Low transportation costs
- Good resistance to chemical attack

- Non-corrosive properties
- Low electrical conductivity
- Low wall friction losses
- Is not prone to encrustation or deposition
- Good flexural strength.
- Can be field jointed and interconnected by electro-fusion welding making it 100% watertight were it not for the risk of third party damage.

PE pipes are manufactured in a fairly limited range of sizes and pressure classes between DE15 and DE355. In the smaller diameters it comes coiled in long lengths ranging from 300 m for diameters between DE15 and DE50 mm to 100 m lengths for DE125 mm. This greatly reduces the need for joints.

In the smaller diameter range it is extremely useful for consumer connections and is now extensively used as such although regrettably has therefore become synonymous with the ubiquitous ‘spaghetti’ distribution. Also unfortunately, polyethylene tubing is also produced for non-pressure applications such as for electrical conduits and where quality control is lacking this gets used for consumer connections often laid with negligible cover with rupture and leakage then a major problem.

For PEHD pipes to DIN 8074:1999 in the diameter range from DE15 to DE50 there are six nominal diameters and above PFA6 there are 5 pressure classes with 29 possible combinations. Between DE63 to DE315 there are 13 nominal diameters and 4 pressure classes (PFA 10, PFA 16, PFA 20 and PFA 25), giving 52 possible combinations. Wall thickness ranges between 1.8 mm (DE15, PFA10), to 5.8 mm (DE63, PFA 10) to 63.2 mm (DE315, PFA 25).

PE pipes are often referred to by Pipe Series (S) or by a Standard Dimension Ratio (SDR). The pipe series is a dimensionless number related to the nominal outside diameter of a pipe, DE, and its wall thickness, s, whilst the standard dimension ratio is the ratio of the outside diameter of a pipe wall to its thickness, calculated as follows:

$$s = DE / (2S + 1) \qquad \text{and,} \quad SDR = 2S + 1$$

Their relationship is as follows where the induced hydrostatic stress, σ =5 N/mm^2.

TABLE 4.43: PE PIPES STANDARD DIMENSION RATIO AND PIPE SERIES

PIPE SERIES (S)	STANDARD DIMENSION RATIO (SDR)	NOMINAL PRESSURE (PFA)
6.3	13.6	8
5.0	11.0	10
3.2	7.4	16
2.5	6.0	20
2.0	5.0	25

As noted above, for pressured water pipelines, it is recommended that PE_{100} with a minimum required strength, (MRS), of 10.0 M/mm^2 be used, where MRS is the resistance to internal hydrostatic pressure in water at 20^{o}C for 50 years.

More detailed information on diameters, and wall thickness and thus on hydraulic capacity, as well as on allowable working pressures is given in the Tables on the following page:

TABLE 4.44: PE PIPE SERIES AND WALL THICKNESS IN mm

DE	PIPE SERIES					
	6.3	5	4	3.2	2.5	2
	NOMINAL PRESSURE IN Bar					
	8	10		16	20	25
16	-	-	1.8	2.2	2.7	3.3
20	1.8	1.9	2.3	2.8	3.4	4.1
25	1.9	2.3	2.8	3.5	4.2	5.1
32	2.4	2.9	3.6	4.4	5.4	6.5
40	3.0	3.7	4.5	5.5	6.7	8.1
50	3.7	4.6	5.6	6.9	8.3	10.1
63	4.7	5.8	7.1	8.6	10.5	12.7
75	5.6	6.8	8.4	10.3	12.5	15.1
90	6.7	8.2	10.1	12.3	15.0	18.1
110	8.1	10.0	12.3	15.1	18.3	22.1
125	9.2	11.4	14.0	17.1	20.8	25.1
140	10.3	12.7	15.7	19.2	23.3	28.1
160	11.8	14.6	17.9	21.9	26.6	32.1
180	13.3	16.4	20.1	24.6	29.9	36.1
200	14.7	18.2	22.4	27.4	33.2	40.1
225	16.6	20.5	25.2	30.8	37.4	45.1
250	18.4	22.7	27.9	34.2	41.6	50.1
280	20.6	25.4	31.3	38.3	46.5	56.2
315	23.2	28.6	35.2	43.1	52.3	63.2

TABLE 4.45: PEHD PIPES ALLOWABLE WORKING PRESSURE

TEMPERATURE ^0C	YEARS OF SERVICE	FACTOR OF SAFETY	PIPE SERIES					
			6.3	5	4	3.2	2.5	2
			ALLOWABLE WORKING PRESSURE (Bar)					
30	25	1.6	8.5	10.8	13.5	16.6	21.2	24.1
	50	2.0	8.4	10.6	13.2	16.3	19.3	23.7
	25	1.6	6.7	8.4	10.6	13.0	16.9	21.2
	50	2.0	6.1	7.7	9.6	11.9	15.4	19.3

For more detailed information, reference should be made to EN 12201:2003, DIN 8074:1999, and DIN 8075:1999.

4.14.3.5 Ferrous Pipes and Their Coatings and Linings

A. Ferrous Pipes

This section refers specifically to Ferrous Pipes (of iron and steel).where corrosion protection is mandatory as both products corrode and evidence suggests at roughly the same rate [< >].

(a) Ductile Iron and DI Pipes

Ductile iron, also called nodular cast iron, is one type of cast iron. Its development dates back to the early 1940’s. The first ductile iron pipes were produced in the mid 1950s and by the mid 1960s it had largely replaced grey cast iron in the pipe industry.

A typical chemical analysis of this material is:

- Carbon 3.3 to 3.8%
- Silicon 2.2 to 2.8%
- Manganese 0.1 to 0.5%
- Sulfur 0.005 to 0.02%
- Phosphorus 0.005 to 0.04%
- Magnesium 0.03 to 0.05%
- Other (small quantities of other elements when required for special applications)
- Iron (i.e., 100% minus the above elements).

Other elements such as copper, tin may be added intentionally to increase tensile and yield strength while simultaneously reducing elongation.

The unique characteristic of ductile iron is that the graphite forms into a spherical shape, instead of irregular flakes as in grey cast iron. Sometimes this is referred to as a "nodular" shape. Ductile iron may also be called "nodular iron". This can be achieved by addition of "nodulizers" into the melt. Yttrium was studied as one of the options.

For corrosion resistant applications 15% to 30% of the Iron in the alloy may be replaced with varying amounts of Nickel and/or Copper and/or Chromium. A recent development in ductile iron metallurgy is Austempered Ductile Iron where the metallurgical structure is manipulated through a sophisticated heat treat process.

Ductile iron (D.I.) pipes are manufactured either as centrifugally cast with sockets and spigots or as double flanged pipes, as well as in a whole range of fittings. Double flanged pipes and fittings are manufactured in four standard pressure classes of PN 10, 16, 25, and 40 whereas spigot and socket pipes are manufactured in three thickness classes with pressure class dependant upon nominal diameter as shown in the table below.

However, the standard thickness class is K9 which will be supplied unless specifically demanded or otherwise required by the Client.

[< >] Tests carried out by the U.S. National Bureau of Standards on uncoated specimens of steel and ductile iron pipe in the same locations have shown both materials corrode at roughly the same rate.
M. Romanoff, “Underground Corrosion”, National Bureau of Standards Circular No. 579, U.S. Government Printing Office, Washington, D.C., 1957

TABLE 4.46: D.I. PIPE DIAMETERS AND PRESSURES (PN)

DN	D.I. PIPE THICKNESS CLASS					
	40		K9		K10	
	PFA	PMA	PFA	PMA	PFA	PMA
40	64	77	85	102	85	102
50	64	77	85	102	85	102
60	64	77	85	102	85	102
65	64	77	85	102	85	102
80	64	77	85	102	85	102
100	64	77	85	102	85	102
125	64	77	85	102	85	102
150	62	74	79	95	85	102
200	50	60	62	74	71	85
250	43	51	54	65	61	73
300	40	48	49	59	56	67
350	40	48	45	54	51	61
400	40	48	42	51	48	58
450			40	48	45	54
500			38	46	44	53
600			36	43	41	49
700			34	41	38	46
800			32	38	36	43
900			31	37	35	42
1000			30	36	34	41
1100			29	35	32	38
1200			28	34	32	38
1400			28	33	31	37
1500			27	32	30	36
1600			27	32	30	36
1800			26	31	30	36
2000			26	31	29	35

Source: EN 545:2002, Table A.1. PFA = allowable operating pressure; PMA = maximum operating pressure (inc surge)

Because of the manufacturing process, D.I. pipes have notably thicker walls than do Steel pipes of the same DN. However and when the minus tolerances and allowance for the brittle white metal surface that develops during the casting process is considered, the actual structural thicknesses for the two ferrous pipes are quite similar.

As a result of both this and the consensus that both materials corrode at approximately the same rate, it has been concluded that the choice between ductile iron and steel pipe in a given situation should be based on factors other than corrosive resistance [< >].

(b) Steel and Steel Pipes

Steel is a metal alloy whose major component is iron, with carbon content between 0.02% and 1.7% by weight. Carbon is the most cost effective alloying material for iron, but many other alloying elements are also used.

< > Pipe Protection, A Review of Current Practice, Page 22 British Hydrodynamics Research Association U.Hassan, C E Jewsbury and A P J Yates, 1978

Carbon and other elements act as a hardening agent, preventing dislocations in the iron atom crystal lattice from sliding past one another. Varying the amount of alloying elements and their distribution in the steel controls qualities such as the hardness, elasticity, ductility, and tensile strength of the resulting steel. Steel with increased carbon content can be made harder and stronger than iron, but is also more brittle.

The maximum solubility of carbon in iron is 1.7% by weight, occurring at 1130° Celsius; higher concentrations of carbon or lower temperatures will produce cementite which will reduce the material's strength. Cast iron with a higher carbon content has a lower melting point. Steel is also to be distinguished from wrought iron with little or no carbon, usually less than 0.035%.

Currently there are several classes of steels in which carbon is replaced with other alloying materials, and carbon, if present, is undesired. Steel can be described as an iron-based alloy that can be plastically formed (pounded, rolled, etc.).

(i) Spiral-welded Steel pipes

When compared to the alternatives such as ductile iron pipe or plastic pipe, spiral welded steel pipe are used for a much wider range of pressures, can attain higher pressure, and are more economic in use of material and hence in energy than D.I.. They also offer more design options, especially for large diameter piping applications.

Spiral welded steel pipe is manufactured from coils of rolled steel. On cost grounds this is usually hot-rolled steel. An automated mill unrolls the coil and forms the steel strip into a tube with a helical seam sealed by a continuous submerged-arc weld, subjected to a continuous X-ray to ensure integrity.

Spiral-weld steel pipes can be supplied to meet any national or international steel pipe standard or manufactured to meet a specific requirement. Pipes can be specifically designed for any project in terms of pipe diameter, length, wall thickness, steel strength, pressure requirements, lining, coating, etc. Pipes are typically produced in an outside diameter ranging from 200mm to 2000 mm. Wall thicknesses vary from 2.3 mm to 20 mm, and standard lengths from 6 m to 12 m, and for pressures ranging from 107.3 Bar to 34.9 Bar. See Table 4.53 for details (ISO and EN steel strengths and thicknesses).

(ii) Longitudinal welded or seamless steel pipes

Electric resistance welded (ERW) pipe is manufactured by cold-forming a sheet of steel into a cylindrical shape. Current is then passed between the two edges of the steel to heat the steel to a point at which the edges are forced together to form a bond without the use of welding filler material. Initially this manufacturing process used low frequency A.C. current to heat the edges. This low frequency process was used from the 1920's until 1970. In 1970, the low frequency process was superseded by a high frequency ERW process which produced a higher quality weld. This method is usually now restricted to diameters of DN300 mm and less.

Double Submerged Arc Welded Steel Pipe manufacturing involves first forming steel plates into cylindrical shapes. The edges of the rolled plate are formed so that V-shaped grooves are formed on the interior and exterior surfaces at the location of the seam. The pipe seam is then welded by a single pass of an arc welder on the interior and exterior surfaces (hence double submerged). The welding arc is submerged under flux.

An advantage of this process is that welds penetrate 100% of the pipe wall and produce a very strong bond of the pipe material.

Seamless Steel Pipe has been manufactured since the 1800's and while the process has evolved, certain elements have remained the same. Seamless pipe is manufactured by piercing a hot round steel billet with a mandrel. The hollowed steel is then rolled and stretched to achieve the desired length and diameter. The main advantage of seamless pipe is the elimination of seam-related defects; however, the cost of manufacture is greater.

Early seamless pipe was susceptible to defects caused by impurities in the steel. As steel-making techniques improved, these defects were reduced, but they have not been totally eliminated. While it seems that seamless pipe would be preferable to formed, seam-welded pipe, the ability to improve characteristics desirable in pipe is limited. For this reason, seamless pipe is currently available in lower grades, wall thicknesses and diameter than welded pipe.

B. Environments (soil categories) for Coating Systems

Soils can be categorised as being class I, II, or III in accordance with the following table [< >]: -

TABLE 4.47: CLASSIFICATION OF SOIL AGGRESSIVENESS

SOIL CATEGORY	SOIL AGGRESSIVENESS	PROBABILITY OF CORROSION	
		WIDE OR DEEP PITTING CORROSION	GENERAL CORROSION
I a	Virtually not aggressive	Very low	Very low
I b	Weakly aggressive	Low	Very low
II	Aggressive	Medium	Low
III	Strongly Aggressive	High	Medium
The soil aggressiveness corresponds to the probability of free corrosion in the absence of extensive concentration cells			

While unalloyed and low-alloy ferrous metals in contact with soil may undergo uniform surface deterioration as a result of corrosion, wide or deep pitting is the rule. Localised corrosion phenomena are generally a result of the formation of corrosion cells or as a result of the action of impressed current cathodes.

Soil categories act as a guide as to the protective coatings acceptable for ductile iron and steel pipes. Ideally tests should be undertaken to determine the aggressiveness of soils where major pipelines and trunk mains are intended, but in the absence of such tests and unless there is specific information to the contrary, a class III soil category should be assumed.

Then only for trunk and primary mains where there is a risk of very aggressive soils, as for example in valley bottoms near the coast or in internal drainage basins where marine shales might be present or in made up ground is it then necessary for thorough investigations.

Guidance on suitable electrically inert, barrier coatings is given in each of the following specific pipe sections.

[< >] Based on DIN 50929 Part 3,Table 2: Classification of soils according to soil aggressiveness and probability of free corrosion of unalloyed and low-alloy ferrous metals.

C. Ferrous Pipes Coatings

(i). Coated D.I. pipes

For ductile iron pipes and where soil tests have not been undertaken, a class III soil category should be assumed for all locations and diameters. Anodic embedment and backfill must be of soil category I and in the case of embedment must meet the embedment class required for the type of soil and its soil modulus (see above). Polyethylene sheet wrapping shall not be used in industrial and urban environments and is probably best avoided altogether. Where it is used then the use of anodic embedment and backfill material is mandatory.

A more recent coating comprising zinc-aluminium having a minimum mass of 400 g/m^2 may be used in class I and II soils [< >].

TABLE 4.48: SOIL CATEGORIES FOR COATED D.I. PIPES

TYPE OF OUTER COATING	COATING THICKNESS IN mm	USE WITH ANODIC EMBEDMENT & BACKFILL	SOIL CATEGORIES
Polyethylene (PE) [factory applied]	1.8 - 3.0 mm	No	I, II, III
Cement mortar (CM)	5.0 mm	No	I, II
Polyethylene sheeting (PES) [use in rural areas only]	0.2 mm	Yes	I, II, III
From DIN 30675:Part 2:1993, Table 1 and sub-clause 4.2			

(ii). Barrier Coated Steel Pipes

Except in the case of diameters currently less than DN80 mm and then only when the pipe is installed within buildings or clear of the ground or in rocky areas or in chambers, barrier coated and not zinc coated (GS) steel pipes shall be considered.

For diameters greater than DN65 mm, and when installed below ground and outside buildings the preferred outermost layer of the coating is polyethylene (PE), used as part of a three-component system. Polyethylene sheeting (PES) should not be used.

TABLE 4.49: SOIL CATEGORIES AND BARRIER COATINGS FOR BURIED STEEL PIPES

TYPE OF COATING	COATING TYPE OR THICKNESS IN µm	CONTINUOUS SERVICE TEMPERATURE (^{0}C)	SOIL CATEGORIES
Polyethylene (PE)	Normal	Up to 50	I, II, III
Epoxy resin powder (EP)	350 350	Up to 30 Up to 50	I, II I, II, III
Polyurethane (PUR)	800 1500 1500	Up to 30 Up to 30 Up to 50	I, II I, II, III I, II
From DIN 30671:1992, Table 1 and DIN 30675:Part 1:1992, Table 1			

< > EN 545-2002, Annexe D, Field of use, characteristics of soils, D.2. However this Standard no longer considers polyethylene wrapping even with anodic backfill in class III soils in rural areas.

For diameters greater or equal to DN80 mm, and when installed above ground and outside buildings the preferred outermost layer of the coating is polypropylene (PP), used as part of a three-component system, as indicated in the following table.

TABLE 4.50: POLYPROPYLENE FOR BARRIER COATED STEEL PIPES

TYPE OF OUTERMOST COATING LAYER	TUBE DIAMETER D (mm)	COATING THICKNESS (mm)	CONTINUOUS CLASS 1 SERVICE TEMPERATURE (^{0}C)
Polypropylene (PP)	D ≤508	1.2	
	508 < D ≤ 762	1.5	Up to 80.
	762 < D	1.7	
From prEN 10286:1996, Table 2 , class 1			

(iii). Galvanised (hot dipped zinc coated) Mild Steel Water Pipes

Historically GS or GI Pipes as they are termed have been used extensively especially in diameters of DN150 mm and less in difficult terrain both above and below ground.

As noted above, strictly speaking such pipes for water supply are not galvanised as galvanising involves the coating of an electrically conductive material such as steel with a layer of metal using an electrical current.

In reality such pipes are ‘hot dipped’ zinc coated steel pipes. This is the process of coating (and lining) steel pipe with a thin zinc layer, by passing the steel through a molten bath of zinc at a temperature of around 460°C. The zinc then "rusts" to form zinc oxide, a fairly strong material that is intended to inhibit further rust, protecting the steel below from the elements. However the outer coating in particular is easily damaged and then the exposed steel corrodes.

Nowadays, it is difficult to find European or North American design manuals, codes of practice or steel pipe specifications that specify zinc as a protective coating for steel pipes to be laid on or below the ground surface. Formerly dealt with in BS 1387:1985 this standard has now been superseded and withdrawn. In part, this is because such a coating has a short guaranteeable working life, usually of only 9 [< >] to 10 years, and is extremely sensitive to pH outside the range from 5.5 to 12.

EN 12502-3:2004, entitled “Protection of metallic materials against corrosion” gives guidance on the assessment of corrosion likely in water distribution and storage systems. Influencing factors for hot dip galvanised as a pipe lining material is discussed and amongst other things it states: -

- There are four types of corrosion of the zinc protective layer namely: uniform corrosion; pitting corrosion; selective; corrosion; and bimetallic corrosion of which high rate uniform corrosion, pitting corrosion and selective corrosion are of concern.
- High rate uniform corrosion occurs the lower the pH, where velocities are very high and there is no possibility of stable protecting rust layers being formed.
- The rate of uniform corrosion decreases in the flow direction as the water is depleted of oxygen and carbon dioxide by the corrosion process.

[< >] “Galvanised pipes are widely used for public & domestic where a 30 micron coating may last up to 9 years depending on the nature of the soil.”
Pipe Protection, A Review of Current Practice Page 30, U. K. V. Hassan, A. Yates & C. Jewsbury, British, Hydrodynamics Research Association–, Nov’1979

- Periods of stagnation, or very low water velocities followed by sudden turbulent flow will hasten the dissolution of corrosion products as will the concentration of carbonic acid species in the water.
- Failure to properly pressure test, flush and commission a new pipeline before putting into service increases the risk of the formation of loosely adherent corrosion products.
- Temperatures in excess of 35°C will hasten pitting corrosion.
- Surface conditions of the bare metal to be coated are important and only Type A (smooth) quality pipes should be used. The inner weld seam of electrically resistance welded pipes is particularly susceptible, especially if they are not diameter-reduced by drawing.
- The rate of corrosion rises if the water contains particulates and particularly if these are hard and abrasive in nature, e.g. fine sand. Also where deposits of particulates form near poorly designed or infrequently operated washouts.

GS pipes have traditionally been jointed by internally threaded sockets and in the process of threading each pipe end to be able to fit the socket the galvanised coating is removed locally and reduces the wall thickness, and increases the risk of corrosion. The use of GS Pipes is not therefore encouraged where suitable alternatives exist (diameters above DN50 mm). If used, only Medium and Heavy Duty classes should be considered and where jointed with threaded sockets is intended, only Heavy Duty class GS pipes are allowed, and each joint area must be additionally protected by the application of either two coats of an approved paint or by a tape wrap. Because GS pipes are not resistant to corrosion in the medium and long term they need an additional protective coating in category II and III soils.

An alternative to GS pipes in the diameter range DN15 mm to DN65 mm is to replace the zinc coating by either a coating of FBE or PE with jointing then made using either similarly protected flexible couplings or flexible grooved joints. PE coated and galvanised lined small diameter steel pipes are now available on the international market and a local steel pipe manufacturer has been requested to give consideration to this.

Where such pipes are to be used in a distribution system, extended flexible grooved joints may be used inclusive of a plugged, threaded boss suitable for a connecting ferrule. However, where flexible groove joints are to be supplied, the manufacturer / supplier must hydraulically test at random, 5% of each joint and diameter before delivery using grooved pipe test pieces of the appropriate diameter, and provide a written test guarantee to this effect.

Maximum working pressures for GS pipes are as given in the following table, (from obsolete BS 1387), but subject to reduction by 40% when threaded joints are used.

TABLE 4.51: MAXIMUM WORKING PRESSURES AND WALL THICKNESS FOR GS PIPES

DN	MAX. WORKING PRESSURE, OUTSIDE DIAMETERS & WALL THICKNESS, t					
	GS MEDIUM CLASS			GS HEAVY DUTY CLASS		
	OD (mm)	(Bars)	t (mm)	OD (mm)	(Bars)	t (mm)
15	21.3	21	2.65	21.3	25	3.25
20	26.9	21	2.65	26.9	25	3.25
25	33.7	21	3.25	33.7	25	4.05
32	42.4	??	3.25	42.4	??	4.05
40	48.3	17.5	3.25	48.3	21	4.05
50	60.3	14	3.65	60.3	17.5	4.50
65	76.1	??	3.65	76.1	14	4.50

D. Ferrous Pipe Linings

Other than galvanising for small diameter steel pipes, ferrous pipe linings include cement mortar for D.I., cement mortar or epoxy for Steel and epoxy-coated cement mortar for both D.I. and Steel.

Because of the minor casting irregularities in the wall of D.I. pipes, it is not possible to ensure a totally corrosion free surface by steel grit blasting so that adhesion between epoxy and the wall cannot be guaranteed. Hence epoxies are not applied directly on to D.I. pipe walls and instead cement mortar or for soft or aggressive waters, epoxy coated cement mortar should be used.

Waters carry varying amounts of different ions resulting from the disassociation of soluble salts found in soils. Waters that have very low ion content are aggressive to calcium hydroxide contained in hydrated cements due to the waters' low content of carbonates and bicarbonates. Soft waters may also have acidic characteristics due to the presence of free CO_2. When cement-mortar linings are subjected to very soft water, calcium hydroxide, $CA(OH)_2$, is leached out. The concentration of leachates increases with the aggressiveness of the water and its residual time in the pipe and is inversely proportional to the diameter of the pipe. These waters will also attack calcium silicate hydrates, which form the larger portion of cement hydrates. Although calcium silicate hydrates are almost insoluble, soft waters can progressively hydrolyse them into silica gels, resulting in a soft surface with reduced mechanical strength.

An epoxy seal-coat retards this leaching and where a pH below 6 is anticipated for the water to be conveyed, a seal coat must be applied [< >], to cement-mortar lined Steel as well as to D.I..

Lining thickness for D.I. pipes are given on the following page in Table 4.50, and for Steel in Table 4.51.

4.14.3.8 Other Pipes

Although far less commonly used or no longer imported, there are several other types of pressure pipes on the international market.

These include: -

- Asbestos Cement Pipes, which due to concerns over the carcinogenic effects of asbestos fibres when mining, pipe manufacturing and laying and cutting are no longer imported. Neither should old pipes be recovered and re-used but instead be very carefully disposed of. Those dismantling and handling such pipes should wear masks over nose and mouth.
- Glass Reinforced Fibre Pipes not suitable for distribution systems due to problems when cutting or making connections but can be considered as a practicable alternative for rural trunk mains but only when required by a project financier.
- Reinforced Concrete Pipes more commonly used in non pressure and low pressure applications.
- Pre-stressed Concrete Pipes which have been manufactured locally and used historically in eastern Africa for major transmission mains. They have been shown to have a limited working life (<30 years) and to require a high to very high level of quality control in

[< >] For example: - Where alum carries over from a treatment works and the water pH drops below 6, a high leaching rate occurs in any cement mortar lined Trunk Mains, the fine sand released from the cement mortar gets carried into the distribution network and into the working mechanism of wet dial consumer water meters with the result that such meters quickly stop recording flow whilst allowing water to pass.

their manufacture. They do however have one particular advantage in that they are not prone to vandalism and illegal connections.

These other pipe types have not been considered further in this version of the Design Manual.

TABLE 4.52: THICKNESS OF CEMENT MORTAR LINING WITH/WITHOUT EPOXY FOR D.I. PIPES

DN	DUCTILE IRON PIPE						
	Pipe OD (mm)	Class No K	Nominal . Iron Thickness (mm)	Nominal Cement Thickness (mm)	Nominal Addit. Epoxy Thickness (µm)	Cement Lined Bore (mm)	Cement & Epoxy Lined Bore (mm)
65	82	9	6.0	3.0	300	64.0	62.8
80	98	9	6.0	3.0	300	80.0	78.8
100	118	9	6.0	3.0	300	100.0	98.8
125	144	9	6.0	3.0	300	126.0	124.8
150	170	9	6.0	3.0	300	152.0	150.8
200	222	9	6.3	3.0	300	203.4	202.2
225	Not Available						
250	274	9	6.8	5.0	300	250.5	249.3
300	326	9	7.2	5.0	300	301.6	300.4
350	378	9	7.7	5.0	300	352.7	351.5
400	429	9	8.1	5.0	300	402.8	401.6
450	480	9	8.6	5.0	300	452.9	451.7
500	532	9	9.0	5.0	300	504.0	502.8
600	635	9	9.9	5.0	300	605.2	604.0
700	738	9	10.8	6.0	300	704.4	703.2
800	842	9	11.7	6.0	300	806.6	805.4
900	945	9	12.6	6.0	300	907.8	906.6
1000	1048	9	13.5	6.0	300	1009.0	1007.8
1200	1255	9	15.3	6.0	300	1212.4	1211.2
1400	1462	9	17.1	9.0	300	1409.8	1408.6
1600	1668	9	18.9	9.0	300	1612.2	1611.0
1800	1875	9	20.7	9.0	300	1815.6	1814.4
2000	2082	9	22.5	9.0	300	2019.0	2017.8

TABLE 4.53: THICKNESS OF EPOXY AND CEMENT MORTAR LINING FOR STEEL PIPES

DN	STEEL PIPE						
	Pipe OD (mm)	Nominal Steel Thickness (mm)	Nominal cement Thickness (mm)	Cement Lined Steel Bore (mm)	Nominal Epoxy Thickness (μm)	Epoxy Lined Bore (mm)	Cement & Epoxy Lined Bore (mm)
65	76.2	2.3	4.0	63.6	350	70.9	62.9
80	88.9	2.3	4.0	76.3	350	83.6	75.6
100	114.3	2.6	4.0	101.1	350	108.4	100.4
125	139.7	2.6	4.0	126.5	350	133.8	125.8
150	168.3	2.6	4.0	155.1	350	162.4	154.4
200	219.1	2.6	4.0	205.9	350	213.2	205.2
225	244.5	3.6	4.0	229.3	350	236.6	228.6
250	273	3.6	4.0	257.8	350	265.1	257.1
300	323.9	4.0	5.0	305.9	350	315.2	305.2
350	355.6	4.0	5.0	337.6	350	346.9	336.9
400	406.4	4.0	5.0	388.4	350	397.7	387.7
450	457.2	5.0	6.0	435.2	350	446.5	434.5
500	508	5.0	6.0	486.0	350	497.3	485.3
600	610	5.6	6.0	586.8	350	598.1	586.1
700	711	6.3	8.0	682.4	350	697.7	681.7
800	813	7.1	10.0	778.8	350	798.1	778.1
900	914	8.0	10.0	878.0	350	897.3	877.3
1000	1016	8.8	10.0	978.4	350	997.7	977.7
1200	1219	10.0	14.0	1171.0	350	1198.3	1170.3
1400	1422	12.5	14.0	1369.0	350	1396.3	1368.3
1600	1626	14.2	14.0	1569.6	350	1596.9	1568.9
1800	1829	14.2	14.0	1772.6	350	1799.9	1771.9
2000	2032	16.0	14.0	1972.0	350	1999.3	1971.3

4.15 INTRODUCTION TO STRUCTURAL DESIGN OF PIPELINES

4.15.1 General Considerations

To be able to carry out the structural design, the Designer should have a thorough knowledge about the pipe materials that are being considered. It. is not sufficient that a particular pipe satisfies his hydraulic design parameters. It should also satisfy all other considerations with the regard to structural integrity, the initial cost, cost of transport, handling, stocking, laying and operation and maintenance.

The Designer should also have a thorough knowledge and cost of all available pipe specials and fittings in all the different pipe materials and these are discussed further in Section 4.17. It is beyond 'the scope of this Manual to list the multitude of specials available but knowledge on this subject can be gained from manufacturers catalogues for the different pipe materials. It is essential to keep all office libraries up to date with these catalogues.

Pipes of the same material in various pressure classes have the same outside diameters so that these can be joined. However different pipe materials have different outside diameters and require different wall thickness to meet the same pressure. This means that internal diameters can vary quite considerably for the same **Nominal Diameter** [< >], and the actual internal diameter of the pipe less that of any lining must be used for flow and friction loss calculations. Because of this where **DN** is used here it refers to D.I. pipes as a standard reference and DE is used for the actual Outside Diameter of the different pipes being considered.

Hence in Table 4.54, the nearest equivalents to D.I. Nominal Diameters are given to enable the Designer to select the closest equivalent.

4.15.2 Working Life of Pipelines

One of the first considerations in any decision making process must be the intended design working life. Except for GS pipelines where effective working life is no more than ten years, it is usual to consider a working life of between 20 years and 40 to 50 years. To minimise complexity when comparing alternatives, it is probably best to consider no more than two options, say of 20 years and 40 years.

Another reason for needing to decide upon working life first is because of age related changes to both pipe materials and internal roughness. If more than one possible working life needs to be considered this then brings with it the need to consider future replacement costs and future disruption to other activities if the different design working life alternatives are to be compared.

Little evaluation seems to have been carried out on the optimisation of working life although in one recent study [< >] which considered several alternatives between 20 and 100 years, a 50-year working life was found to be optimum.

This then suggests that for urban areas and trunk mains at least, the design working life of a pipeline should not be less than 40 years. The Designer should therefore advise the Client accordingly.

4.15.3 Comparison of Pipe Wall Diameters and Wall Thickness Tolerances

As noted above, before proceeding to the structural design requirements for different pipelines, it is important that Designers fully understand and allow for the differences in internal diameter (ID) when specifying pipes by their nominal diameter. It is also necessary when carrying out a structural design that there is an appreciation of differences in wall thickness minus tolerances which are largely a result of manufacturing processes, particularly for D.I. and Steel.

Due to its manufacturing process by centrifugal casting, D.I. pipes have the largest minus tolerances, followed by steel pipes made from hot rolled plate, then steel pipes made from cold rolled plate and then the thermoplastic pipes which due to the extrusion process have the smallest minus tolerances.

< > For example: - A DN200 D.I. pipe with cement-mortar lining has an internal diameter of 203.4 mm, a DN200 steel pipe with cement-mortar lining has an internal diameter of 199.9 mm but this is 211.2 mm if the internal lining is epoxy. The nearest thermoplastic equivalents are a DN225 PN12.5 PVCu pipe with an internal diameter of 203.4mm and a DN250 PEHD PN12.5 pipe with an internal diameter of 204.5 mm.

< > Life-Cycle Energy Analysis of a Water Distribution System, Yves R. Filion,1 Heather L. MacLean,2 A.M.ASCE; and Bryan W. Karney,3 M.ASCE, J. Infrastruct. Syst., Volume 10, Issue 3, pp. 120-130 (September 2004),

In addition to the casting tolerance, D.I. has an extra thickness due to the formation of a brittle white iron surface that develops during casting and this is not to be taken into account in pressure calculations.

As the vast majority of steel pipes are manufactured from hot rolled steel plate which is the cheaper of the two alternatives, Table 4.52 on the following page provides a comparison between D.I. and Steel pipes made from hot rolled steel plate.

Then Table 4.53 compares wall thickness and pressures and pressure classes for Steel pipes according to ISO 559 and EN 10224 minimum thickness. As a general rule, Steel pipes with wall thickness to EN 10224 and either cement mortar or epoxy lining will be found suitable for diameters up to DN300 and to ISO 559 with epoxy lining for diameters larger than that.

Whilst the minus tolerance and hence minimum thickness is important when determining strength and pressure, for hydraulic design the nominal internal diameter (ID) should be used. These are shown in Table 4.54.

4.15.4 Materials Data Required

Materials data for all pipes is required to enable structural design to proceed. Common to all is the internal diameter, excluding lining (D_i) and where applicable including lining (D_{Int}), and the wall thickness (t) being considered. Flexible pipes also require the stiffness, [$S=E \times I/D^3$] to be taken into account.

In addition, steel requires knowledge of the maximum allowable working pressure, PN, the maximum allowable surge pressure, Pt, the allowable stress (f_a) and the allowable deflection $(\Delta/D)_A$, whilst PEHD and PVCu require knowledge of long term ring bending modulus of elasticity (E_L), the allowable deflection $(\Delta/D)_A$, and the allowable long term combined stress (f_a).

4.15.5 Embedment Data Required

The embedment information required and as discussed in Section 4.14.2.2, is primary to the design process. Whilst the designer cannot vary the native soil, embedment characteristics can be modified to obtain a satisfactory and economic solution. These include (within limits) the trench width, B, the type of bed and surround materials and the degree of compaction.

To be remembered in this regard is the requirement regarding the use of anodic embedment and backfill material for D.I. pipes in rural areas and in class III soils where polyethylene sleeving (PUS) is being considered.

4.15.6 Loading and Other Data Required

Loading data requirements and as discussed in Section 4.14.2.3 includes the minimum and maximum depths of cover, H_{min} and H_{max}, and its unit weight, γ_s, as well as the type of surface surcharge and hence its load, p_s and the normal maximum operating pressure in the pipeline, p_i (or PFA).

In addition, it is necessary for PVC pipeline design to know the likely upper temperature of the water to be conveyed and to estimate the daily frequency of hydrodynamic fatigue effects from all sources (vehicle impacts, valve operation) if this is expected to exceed 10 occurrences daily.

TABLE 4.54: COMPARISON OF WALL THICKNESS AND TOLERANCES, STEEL AND D.I. PIPES

DN	Steel Pipe DIN 2460						Ductile Iron Pipe DIN 28600					DI thick- ness more than STEEL by
	PN Steel St37 (Bar)	PN Steel St44 (Bar)	PN Steel St52 (Bar)	Nom. wall thick- ness (mm)	less minus tolerance DIN 1626 (mm)	usable wall thick- ness (mm)	PN (Bar)	Nom. thick- ness K=9 (mm)	less minus tolerance CL. 4.3 (mm)	less casting skin CL. 3.1 (mm)	usable wall thick- ness (mm)	(%)
80	100	100	125	3.2	0.35	**2.9**	40	5.2	1.38	1.5	**2.3**	-23%
100	63	80	100	3.2	0.35	**2.9**	40	5.4	1.4	1.5	**2.5**	-14%
150	50	63	80	3.6	0.35	**3.3**	40	5.9	1.45	1.5	**3**	-10%
200	50	50	63	3.6	0.35	**3.3**	40	6.3	1.5	1.5	**3.3**	2%
250	40	40	50	4	0.35	**3.7**	40	6.8	1.55	1.5	**3.8**	3%
300	40	40	50	4.5	0.35	**4.2**	40	7.2	1.6	1.5	**4.1**	-1%
350	32	40	50	4.5	0.35	**4.2**	40	7.7	1.65	1.5	**4.6**	9%
400	32	40	50	5	0.35	**4.7**	32	8.1	1.7	1.5	**4.9**	5%
450	32	40	50	5.6	0.35	**5.3**	32	8.6	1.75	2.5	**4.4**	-21%
500	32	32	40	5.6	0.35	**5.3**	32	9	1.8	1.5	**5.7**	8%
600	25	32	40	6.3	0.35	**6**	32	9.9	1.9	1.5	**6.5**	8%
700	25	25	32	6.3	0.35	**6**	32	10.8	2	1.5	**7.3**	18%
800	25	25	32	7.1	0.35	**6.8**	25	11.7	2.1	1.5	**8.1**	17%
900	25	25	32	8	0.35	**7.7**	25	12.6	2.2	1.5	**8.9**	14%
1000	25	25	32	8.8	0.35	**8.5**	25	13.5	2.3	1.5	**9.7**	13%
1200	25	25	32	11	0.5	**10.5**	25	15.3	2.5	1.5	**11.3**	7%
1400	25	25	32	12.5	0.5	**12**	25	17.1	2.7	1.5	**12.9**	7%
1600	25	25	32	14.2	0.5	**13.7**	25	18.9	2.9	1.5	**14.5**	6%
1800	25	25	32	16	0.5	**15.5**	25	20.7	3.1	1.5	**16.1**	4%
2000	25	25	32	17.5	0.5	**17**	25	22.5	3.3	1.5	**17.7**	4%

DI minus tolerance = 1.3 + 0.001xDN according to DIN 28600 Clause 4.3: Casting skin = 1.5mm according to DIN 28600 Clause 3.1

TABLE 4.55: STEEL PIPE WALL THICKNESS AND WORKING PRESSURES

		ISO 559 Steel Water Pipe Thicknesses and Working Pressures																EN 10224			
Nom. Dia. DN	Outside Dia. OD	Series B				Series C				Series D				Series E				Minimum Allowable Wall Thicknesses			
		Wall Thick-ness	Yield Sress (MPa) 235	275	355	Wall Thick-ness	Yield Sress (MPa) 235	275	355	Wall Thick-ness	Yield Sress (MPa) 235	275	355	Wall Thick-ness	Yield Sress (MPa) 235	275	355	Wall Thick-ness	Yield Sress (MPa) 235	275	355
			Working Pressure				Working Pressure				Working Pressure				Working Pressure				Working Pressure		
mm	mm	mm	bars	bars	bars	mm	bars	bars	bars	mm	bars	bars	bars	mm	bars	bars	bars	mm	bars	bars	bars
65	76.1	2.3	71.0	83.1	107.3	2.6	80.3	94.0	121.3	2.6	80.3	94.0	121.3	2.9	89.6	104.8	135.3	2.0			
80	88.9	2.3	60.8	71.1	91.8	2.9	76.7	89.7	115.8	2.9	76.7	89.7	115.8	3.2	84.6	99.0	127.8	2.0			
100	114.3	2.6	53.5	62.6	80.8	2.9	59.6	69.8	90.1	3.2	65.8	77.0	99.4	3.6	74.0	86.6	111.8	2.0			
125	129.7	2.6	43.7	51.2	66.1	3.2	53.8	63.0	81.3	3.6	60.6	70.9	91.5	4.0	67.3	78.7	101.6	2.0			
150	168.3	2.6	36.3	42.5	54.8	3.2	44.7	52.3	67.5	4.0	55.9	65.4	84.4	4.5	62.8	73.5	94.9	2.0			
200	219.1	2.6	27.9	32.6	42.1	3.6	38.6	45.2	58.3	4.5	48.3	56.5	72.9	6.3	67.6	79.1	102.1	2.0			
250	273.0	3.6	31.0	36.3	46.8	4.0	34.4	40.3	52.0	5.0	43.0	50.4	65.0	6.3	54.2	63.5	81.9	2.0			
300	323.9	4.0	29.0	34.0	43.8	4.5	32.6	38.2	49.3	5.6	40.6	47.5	61.4	7.1	51.5	60.3	77.8	2.6			
350	355.6	4.0	26.4	30.9	39.9	5.0	33.0	38.7	49.9	5.6	37.0	43.3	55.9	8.0	52.9	61.9	79.9	2.6			
400	406.4	4.0	23.1	27.1	34.9	5.0	28.9	33.8	43.7	6.3	36.4	42.6	55.0	8.8	50.9	59.5	76.9	2.6			
450	457.2	4.0	20.6	24.1	31.1	5.0	25.7	30.1	38.8	6.3	32.4	37.9	48.9	10.0	51.4	60.1	77.6	2.9			
500	508.0	5.0	23.1	27.1	34.9	5.6	25.9	30.3	39.1	6.3	29.1	34.1	44.0	11.0	50.9	59.5	76.9	2.9	Individual Case Design Situation		
600	610.0	5.6	21.6	25.2	32.6	6.3	24.3	28.4	36.7	6.3	24.3	28.4	36.7	12.5	48.2	56.4	72.7	2.9			
700	711.0	6.3	20.8	24.4	31.5	7.1	23.5	27.5	35.5	7.1	23.5	27.5	35.5	14.2	46.9	54.9	70.9	4.5			
800	813.0	7.1	20.5	24.0	31.0	8.0	23.1	27.1	34.9	8.0	23.1	27.1	34.9	16.0	46.2	54.1	69.9	4.5			
900	914.0	8.0	20.6	24.1	31.1	8.8	22.6	26.5	34.2	10.0	25.7	30.1	38.8	18.5	47.6	55.7	71.9	4.5			
1000	1016.0	8.8	20.4	23.8	30.7	10.0	23.1	27.1	34.9	11.0	25.4	29.8	38.4					4.5			
1100	1118.0	8.8	18.5	21.6	27.9	10.0	21.0	24.6	31.8	11.0	23.1	27.1	34.9					5.0			
1200	1219.0	10.0	19.3	22.6	29.1	11.0	21.2	24.8	32.0	12.5	24.1	28.2	36.4					5.0			
1400	1422.0	12.5	20.7	24.2	31.2	14.2	23.5	27.5	35.5	14.2	23.5	27.5	35.5					5.6			
1600	1626.0	14.2	20.5	24.0	31.0	16.0	23.1	27.1	34.9	16.0	23.1	27.1	34.9					6.3			
1800	1829.0	14.2	18.2	21.4	27.6	16.0	20.6	24.1	31.1	17.5	22.5	26.3	34.0					7.1			
2000	2032.0	16.0	18.5	21.7	28.0	17.5	20.2	23.7	30.6	20.0	23.1	27.1	34.9					8.0			
																		From Table 4			

Note: Intermediate non-standard diameters, e.g. DN225, DN550, DN750 can be produced to order.

TABLE 4.56: COMPARISON BETWEEN NOMINAL (DN) AND INTERNAL DIAMETERS (ID)

DN FOR DI STEEL	DI PIPE K9 O.D.	DI PIPE K9 PIPE THICK EN545	DI PIPE K9 I.D.	DI PIPE K9 CEMENT MORTAR DIN2614 NOM Note 1	DI PIPE K9 CEMENT MORTAR DIN2614 MIN Note 1	DI PIPE K9 CEMENT LINED D.I. NOM I.D.	STEEL PIPE O.D.	STEEL PIPE PIPE THICKNESS EN 10224	STEEL PIPE PIPE THICKNESS ISO559 class	STEEL PIPE I.D.	STEEL PIPE CEMENT MORTAR DIN2614 NOM Note 1	STEEL PIPE CEMENT MORTAR DIN2614 MIN Note 1	STEEL PIPE CEMENT LINED STEEL NOM I.D.	STEEL PIPE EPOXY LINED STEEL I.D.	PVCu PIPE PN 10.0 PIPE DN O.D.	PVCu PIPE PN 10.0 Note	PVCu PIPE PN 10.0 MIN THICK EN 1452-2	PVCu PIPE PN 10.0 I.D.	PVCu PIPE PN 12.5 PIPE DN O.D.	PVCu PIPE PN 12.5 Note	PVCu PIPE PN 12.5 MIN THICK EN 1452-2	PVCu PIPE PN 12.5 I.D.	PEHD PN 10 PIPE DN O.D.	PEHD PN 10 I.D.	PEHD PN 12.5 PIPE DN O.D.	PEHD PN 12.5 MIN THICK	PEHD PN 12.5 I.D.
	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)		(mm)	(mm)	(mm)	(mm)	(mm)	(mm)		(mm)	(mm)	(mm)		(mm)	(mm)	(mm)	(mm)	(mm)		(mm)
60	77	6.0	65.0																				75	**61.364**			
65	82	6.0	70.0			**60.0**	76.1	2.9		70.3	5.0	3.0	**60.3**	**69.6**	75	2.5	3.6	**67.8**					90	**73.636**			
80	98	6.0	86.0	3.0	1.5	**80.0**	88.9	3.2		82.5	5.0	3.0	**72.5**	**81.8**	90	2.5	4.3	**81.4**					90	**73.636**			
100	118	6.0	106.0	3.0	1.5	**100.0**	114.3	3.2		107.9	5.0		**97.9**	**107.2**	110	2.0	4.2	**101.6**							110	10.0	**90.0**
															125	2.0	4.8	**115.4**	125	2.0	6.0	**113**			140	12.7	**114.5**
125	144	6.0	132.0	3.0	1.5	**126.0**	139.7	3.6		132.5	5.0	3.0	**122.5**	**131.8**	140	2.0	5.4	**129.2**	140	2.0	6.7	**126.6**			160	14.5	**130.9**
															160	2.0	6.2	**147.6**	160	2.0	7.7	**144.6**			180	16.4	**147.3**
150	170	6.0	158.0	3.0	1.5	**152.0**	168.3	3.6	C-D	161.1	5.0	3.0	**151.1**	**160.4**	180	2.0	6.9	**166.2**	180	2.0	8.6	**162.8**			200	18.2	**163.6**
															200	2.0	7.7	**184.6**	200	2.0	9.6	**180.8**			225	20.5	**184.1**
200	222	6.3	209.4	3.0	1.5	**203.4**	219.1	3.6	C	211.9	6.0	4.0	**199.9**	**211.2**	225	2.0	8.6	**207.8**	225	2.0	10.8	**203.4**			250	22.7	**204.5**
															250	2.0	9.6	**230.8**	250	2.0	11.9	**226.2**			280	25.5	**229.1**
225			Not Available				244.5				6.0	4.0	**229.3**	**243.8**													
250	274	6.8	260.4	3.0	1.5	**254.4**	273.0	4.0		265	6.0	4.0	**253.0**	**264.3**	280	2.0	10.7	**258.6**	280	2.0	13.4	**253.2**			315	28.64	**257.7**
300	326	7.2	311.6	3.0	1.5	**305.6**	323.9	4.5		314.9	6.0	4.0	**302.9**	**314.2**													
350	378	7.7	362.6	5.0	2.5	**352.6**	355.6	4.5	B-C	346.6	7.0	5.0	**332.6**	**345.9**													
400	429	8.1	412.8	5.0	2.5	**402.8**	406.4			396.4	7.0	5.0	**382.4**	**395.7**													
450	480	8.6	462.8	5.0	2.5	**452.8**	457			447	7.0	5.0	**433**	**446.3**													
500	532	9.0	514.0	5.0	2.5	**504.0**	508	5.6		496.8	7.0	5.0	**482.8**	**496.1**													
600	635	9.9	615.2	5.0	2.5	**605.2**	610	6.3		597.4	7.0	5.0	**583.4**	**596.7**													
700	738	10.8	716.4	6.0	3.0	**704.4**	711	6.3		698.4	9.0	6.0	**680.4**	**697.7**													
800	842	11.7	818.6	6.0	3.0	**806.6**	813	7.1		798.8	9.0	6.0	**780.8**	**798.1**													
900	945	12.6	919.8	6.0	3.0	**907.8**	914			898	9.0	6.0	**880.0**	**897.3**													
1000	1048	13.5	1021.0	6.0	3.0	**1009.0**	1016	8.8		998.4	13.0	10.0	**972.4**	**997.7**													
1100	1152	14.4	1123.2	6.0	3.0	**1111.2**	1118	8.8		1100.4	13.0	10.0	**1074**	**1099.7**													
1200	1255	15.3	1224.4	6.0	3.0	**1212.4**	1219	11		1197.0	13.0	10.0	**1171.0**	**1196.3**													
1400	1462	17.1	1427.8	9.0	4.0	**1409.8**	1422	12.5		1397.0	15.0	12.0	**1367.0**	**1396.3**													
1500	1565	18.0	1529.0	9.0	4.0	**1511.0**																					
1600	1668	18.9	1630.2	9.0	4.0	**1612.2**	1626	14.2		1597.6	15.0	12.0	**1567.6**	**1596.9**													
1800	1875	20.7	1833.6	9.0	4.0	**1815.6**	1829	16		1797.0	15.0	12.0	**1767.0**	**1796.3**													
2000	2082	22.5	2037.0	9.0	4.0	**2019.0**	2032	17.5		1997.0	15.0	12.0	**1967.0**	**1996.3**													

Note 1: According to Procedure 1. Note 2: Design Coefficient C.

SDR = DN/e, where e = wall thickness
Nominal wall thicknesses conform to ISO 4065:1996.
To apply an overall service (design) coefficient of 2.5 (instead of 2.0) for PVCu pipes with DN above

4.16 STRUCTURAL DESIGN OF PIPELINES

The structural design of Pipelines is considered in the following sections. Each case is presented in the form of a design flow diagram to help in the understanding. In the case of thermoplastic and steel pipes worked examples are also included whilst design spreadsheets that can facilitate the computations are illustrated.

The principle reference sources are the WRc Pipe Materials Selection Manual and the requirements of UK National Annex A in EN1295-1:1998 for the Structural Design of Buried Pipelines under Various Conditions of Loading. Where, in a few instances both of these are silent on specific values, reference has also been made to the AWWA M11 (2004) Steel Pipe – A Guide for Design and Installation, CP BS 2010-2, Pipelines, Design and Construction of Steel Pipelines in Land (1970) for Steel Pipes, the WRc, uPVC Pipe Selection Manual (2002) and ISO 4442 for PVCu Pipes.

4.16.1 Design Equations Used

The following equations are used in the structural design of the pipelines considered:

1. Overburden (Soil Load) Pressure at minimum cover, P_{emin}

$$P_{emin} = \gamma_s \times H_{min} \quad N/m^2 \qquad 4.1$$

Where,

γ_s = Soil Density in N/m^3

H_{min} = Depth of Minimum Cover to top of pipe in m

2. Surcharge from Traffic Load at minimum cover, P_s

A modified Boussinesque's equation is used in the WRc Manual to distribute the traffic load surcharge. The values quoted here in Table 4.38 exclude any in-built Traffic Impact Factor (1.5 in the WRc Manual), to give the Designer flexibility in making the selection.

The values presented in Table 4.38 should therefore be multiplied by the Traffic Impact Factor (= 1.0 for parked vehicles) and the Traffic Overload Factor. Values for Traffic Impact Factor lie between 1.5 and 2.0 depending on road surface with a minimum of 1.67 recommended. For Traffic overload, a minimum factor of 1.2 is recommended.

The basic values can alternatively be represented for the three cases of Major Road, Normal Road and Off-road (field) Conditions by the following equations:

Major Road: $49.743 \times H^{-1.0538}$ 4.2

Normal Road: $40.600 \times H^{-1.6809}$ 4.3

Off Road: $22.800 \times H^{-1.5375}$ 4.4

3. Max. Allowable Working Pressure (steel pipes), PN

This is Barlow's Hoop Stress Formula with Utilisation and Joint Factors introduced.

$$PN = (20 \times \sigma_y \times F_{wp} \times t \times V_n)/DE \qquad 4.5$$

Where,

σ_y = Minimum yield stress of the selected steel in MPa

F_{wp} = Working pressure utilisation factor = 0.50 (AWWA-M11)

t = Wall thickness selected (usually from ISO 559 or EN 10224) in mm

V_n = Joint factor/welding efficiency (BS CP 2010: 1.0 for spiral welded steel pipe)

DE = Outside diameter in mm.

Should none of the combinations of Yield Stress and ISO/EN wall thickness meet the requirement, Designers should approach the Steel Pipe manufacturer regarding the use of steels with higher yield stress and/or other possible wall thickness.

4. Specified Yield Strength (steel pipes), P_t

$P_t = (20 \times \sigma_y \times t \times V_n)/DE$ in Bars 4.6

5. Vacuum Load, P_n (all pipes)

Because the pipeline carries water, it is not possible to get a condition of total vacuum. When the hydraulic grade line drops below the elevation of the pipe due to surge effects, water vapour is formed in the pipe. Vapour pressure is 0.2 Bar absolute. Hence the maximum Vacuum Load to be allowed is 1.0-0.2 = 0.8 Bar or 80,000N/m^2.

6. Pipe Modulus and Permissible Stresses

For **ferrous pipes**, the Short-term and Long-term Elastic Modulus are the same.

For D.I. a value of 170,000 MPa should be used and for Steel a value of 207,000 MPa.

For **PEHD** and **PVCu** [<>], the Short-term Elastic Modulus may be taken as 680 - 600 MPa and 2,800 MPa respectively. The Long-term value for PVC = 504 MPa.

However for **PVC pipes**, (but NOT PEHD pipes), there is need to re-rate the pipe to obtain the actual Design Stress values taking into account, design working life, temperature of water being conveyed, and the likely average number of daily hydrodynamic (fatigue causing) impacts due to surcharge impacts and positive and negative surge impacts. (vehicle impacts, valve operation, positive & negative surge)

Whilst working life and water temperature can be reasonably well determined, deciding upon likely average long term values of fatigue-causing impacts is difficult and all a Designer can do is to consider the amount of heavy traffic likely to use the road or that might drive onto the road verge above the pipeline during its working life and select the best value considered to be realistic. If a value commensurate with road verge traffic movement is selected, there is then a case for considering the use of either the next higher pressure class (wall thickness) for road crossings or providing sleeving for such locations.

In the equations, a minimum of 10 daily hydrodynamic occurrences is assumed, resulting in a fatigue impact factor of 1.0.

Even if the required design working life is 40 years, it is suggested that a 50 year life be considered, this being the basis for the guide values given in the various Standards.

For **PVC pipes**, the Minimum Circular Wall Stress @20 deg C and 50 years, f_{circ} is 25 MPa.

The Factors of Safety (FoS) for **PVC pipes** are 2.5 for OD <= 90 and 2.0 for OD > 90:

The equations applied are:

[<>] At 30^{o}C with pipe under sustained pressure, short term modulus = 600 MPa whilst at 20^{o}C = 680 MPa.

i) No hydrodynamic effects @ Operating Water Temp (T_{op}) in ^oC:

Max Permissible Design Stress, = (f_{circ}/FoS) × (1-y × (T_{op} – 20)) 4.7

Where y= 0.02 when ambient temperatures >= 20^oC and = 0.015 when < 20^oC.

ii) With hydrodynamic effects @ Operating Temp:

Max Permissible Design Stress, = (Value from Eq. 4.7)/(R_R) 4.8

Where R_R, the Fatigue re-rating factor, may be obtained from the expression:

$R_R = 0.5039 \times (\text{Ave. No. of Daily Occurrences})^{0.2917}$ (R_R = 1.0 for 10 or less) 4.9

At the Tanzanian Coast, maximum water temperatures in buried pipelines have been measured in excess of 30^oC and this is the value recommended near the Coast. Away from the Coast but subject to ambient temperature, a value of 25^oC may be permissible.

7. Pipe Stiffness, S

For Ferrous Pipes, the Initial (S_i) and Long-term (S_L) Stiffness are given by:

$S_i = E_i \times ((t-t')/(DE-(t-t')))^3/12 + E_c \times (t_c/(DE-(t-t')-t_c))^3/12$ (in N/m^2) 4.10

$S_L = E_L \times ((t-t')/(DE-(t-t')))^3/12 + E_c \times (t_c/(DE-(t-t')-t_c))^3/12$ (in N/m^2) 4.11

For PVC and PE where there is no cement mortar lining (E_c = 0), these simplify to:

$S_i = Ei \times ((t-t')/(DE-(t-t')))^3/12$ (in N/m^2) 4.12

$S_L = E_L \times ((t-t')/(DE-(t-t')))^3/12$ (in N/m^2) 4.13

Where,

t = Mean pipe wall thickness in mm
t' = Minus wall thickness tolerance in mm
t_c = Minimum thickness of cement mortar lining, (ferrous pipes only) in mm
$t^3/12$ = I, the Second Moment of area of pipe wall per unit length
E_i = The Short Term Elastic Modulus, and
E_L = The Long Term Elastic Modulus

8. Native Soil Modulus, E'_3 in MPa selected from Table 4.32

The Native Soil Modulus is selected from Table 4. 32.

9. Soil Modulus of Pipe Bed and Surround, E'_2 selected from Table 4.37

The Embedment Class and Degree of Compaction are selected first and with this input data, the resulting value of Pipe Surround Modulus is found from Table 4.37.

10. Effective overall soil modulus and C_L, the coefficient needed for the calculation

$$E' = E'_2 \times C_L \qquad 4.14$$

Where,

$$C_L = \frac{0.985 + 0.544\ B/DE}{(1.985 - 0.456\ B/DE)E'_2/E'_3 - (1-B/DE)} \qquad 4.15$$

And,

B = Trench Width, in mm, and

DE = Pipe Outside Diameter in mm (**Note:** For ferrous pipes this is NOT DN)

If the native soil modulus is greater than or equal to 5 MN/m^2 or if the trench width is more than 5 times the pipe diameter, the value of E' can be taken as the modulus of the pipe bed and surround E'_2. In flexible pipes the properties of the bottom and lower backfill and sidefill are usually more important that the properties of the native soil.

11. Relative Deflection, Δ/D and Crown and Spigot Deflection Checks, (Ovalization)

The initial deflection $\Delta/D = [k_x \times (p_e + p_s)] / [8S_i + 0.061E']$ 4.16

The long term deflection $\Delta/D = [k_x \times (D_L \times p_e + p_s) \times D_R] / [8S_i + 0.061E']$ 4.17

Where,

k_x = Deflection (bedding) coefficient from Table 4.37

D_L = Deflection lag factor from Table 4.37

D_R = Re-rounding factor = $(1 - p_i / 40)$ 4.18

Where,

p_i = the allowable operating pressure is in Bar, but is only applied if cover depth does not exceed 2.5 m.

E' = Effective overall modulus of soil reaction (see 10 below)

For spigots in flexibly jointed spigot and socket pipes, it is recommended that vertical spigot deflection should be limited to 2% so as to avoid leaks at flexible joints due to excessive deflection and ovalization.

12. Max. Positive Surge Pressure, P_m, and Oversurge Amplitude

Surge due to sudden arrest of the water column *(Joukovsky Pressure Rise)* is:

$$P_m = \frac{V}{g} \times \frac{c_w}{\sqrt{(1 + (D/t) \times (E_w/E))}} \quad \text{(m)} \qquad 4.19$$

Where,

c_w = Celerity of pressure wave in a column of water = 1425 m/s

E_w[< >] = Bulk Modulus of Water in MPa (=2,210 MPa @ 25°C & 2,230 MPa @ 30°C)

E = Elastic Modulus of pipe wall in MPa

t = Wall thickness in mm

D = Mean Diameter in mm (= outside diameter - wall thickness)

V = Flow Velocity in m/s

This is the maximum surge value that is likely to occur and satisfying this requirement in the equations below will ensure that the maximum possible positive surge pressure can be handled. It may be used initially for trunk mains and pipe diameters of DN300 and greater but can be reduced by a factor of 0.75 for smaller pumping mains and 0.5 in

[< >] For water at about 5°C the bulk modulus =2070 MPa and the elastic modulus of steel = 207,000 MPa. Hence E_w/E is then 1/100 for steel which is used in AWWA M11 without explanation as to its derivation.

gravity and non-pumping distribution systems where only valve operation or pipeline burst is likely to cause surge pressure waves.

For **Steel** pipes the maximum allowable pressure inclusive of surge (PMA), should not exceed 0.75 × σ_y, the specified yield strength, and for all flexible pipes:

Over surge pressure (amplitude) = (2 × P_m) must be <= PFA/2 4.20

Where failure occurs in one or both of these equations and especially for pumping mains with DE > 300, a surge analysis is mandatory as the use of proper anti-surge devices will usually then enable the equations to be satisfied.

13. Combined or Hoop Tensile Stress, f_c or f_h, for actual working pressure

For **PVC and PE** pipes, the combined stress has to be calculated:

f_c = $(P_i - P_e)$ DE / 2t + $E_L \times D_f \times \Delta$/DE × t/DE which must not exceed f_a 4.21

Where,

D_f = the Strain Factor from Table 4.37.

If daily fatigue re-rating frequency > 10 (Re-rating factor>1) then put $P_e = 0$

For **Steel** pipes, only the hoop stress is needed:

f_h = $(P_i - P_e)$ DE / 2t which must not exceed f_a 4.22

14. Buckling Stability Check, (p_{crl} and p_{cls})

The critical short & short term external pressures causing buckling of a pipe are given by:

$p_{crs} = 24 \times F_s \times S_i$ and $p_{crl} = 24 \times F_s \times S_L$ 4.23 & 4.24

Where,

F_s = the support factor and is taken as:

= 1.0 for pipes without soil support and when the soil cover is less than 1.5m (this simulates a condition when the soil surrounding adjacent to pipes may be temporarily removed due to installation of other services). In such a case, there is no traffic load.

= $0.025\ (E'/S)^{0.67}$ for pipes where soil cover is of 1.5m or greater 4.25

For **Steel** Pipes, $p_{crl} = p_{crs}$ as $S = S_i = S_L$

The Factor of Safety, FoS, against buckling for **all flexible pipes** is given by:

FoS = $1/(P_e/p_{crl} + (P_s + P_n)/p_{crs})$, (minimum value must be considered) 4.26

The maximum vertical surcharge pressure (P_s) and the internal vacuum pressure during surge (P_n) both occur but only for very short durations, and the probability of them occurring simultaneously is unlikely such that it may be ignored. It is therefore more realistic to design for one alone, (P_s or P_n), whichever is the more severe.

For H < 1.5m, and where Fs = 1, the loads to be applied when checking for the factor of safety against buckling are the overburden (soil) load, (P_e) and the vacuum load (P_n) as it may be assumed that there will be no surcharge (vehicle) load, P_s at the time.

15. De-rated Working Pressure for PVC Pipes, (p_{der})

For PVC Pipes it is necessary to confirm that the de-rated working pressure is sufficient for the actual working pressure using the expression:

p_{der} = 20 × (Max. allowable σ_s @ T_{op}) × t / (DE-t) and this must be >= to PN 4.27

Semi-rigid pipes (D.I.)

It will rarely be necessary to undertake a structural design for D.I., except where either large diameter pipes are contemplated or normal pressures approach the limits as given in Table 4.44. Then, trench conditions can become important. However because narrow trench conditions may not be achieved on site, only wide trench theory should be adopted. (Section 16 for reference purposes only)

16. Design overburden pressure (embankment and wide trench conditions)

For Embankment and Wide Trench conditions, the design overburden pressure is the lower of the values given in the following equation with C_1 calculated for both the "complete" and "incomplete" projection conditions.

$p_e = C1 \times \gamma \times H$ for "complete" projection conditions 4.28

Where,

$C_1 = (e^x - 1)/x$, and 4.29

$x = 2K \times \mu' \times H/D_0 = 2 \times 0.190 \times H/D_0$ for all practical purposes 4.30

Where,

μ' = Coefficient of friction at trench wall

For "incomplete" projection conditions, C_1 is a function of the settlement-deflection and projection ratio, and

$C_1 = 1 + 0.585 (1-n)^{0.48} - 0.0875 (1-n) D_0/H$ 4.29

Where,

n = Pipe-soil stiffness factor

17. Relative Vertical Deflection, (Ovalization), Δ/DE

The initial deflection $\Delta/DE = [k_x \times (p_e + p_s)] / [8S + 0.061E']$ 4.30

The long term deflection $\Delta/DE = [k_x \times (D_L \times p_e + p_s) \times D_R] / [8S + 0.061E']$ 4.31

18. Pipe Wall Bending Stress, f_b

The bending stress (f_b) in the walls of semi-rigid pipes is calculated as follows:

$f_b = E \times D_f (\Delta/DE) \times (t/DE)$ 4.32

However,

1. This value need only be calculated where the length/diameter ratio ≥ 25.
2 For D.I. pipes of K9 or higher, the value of D_f can be taken as 3.5 in all cases.
3. The stress should be calculated for both the initial and long term deflections.

The calculated bending stress, f_b, should not exceed the allowable bending stress, f_{ba}.

The symbols used are summarised on the following page with worked examples illustrating the use of the use of these formulae for PVCu and Steel pipes given on subsequent pages.

Symbols Used and their Units

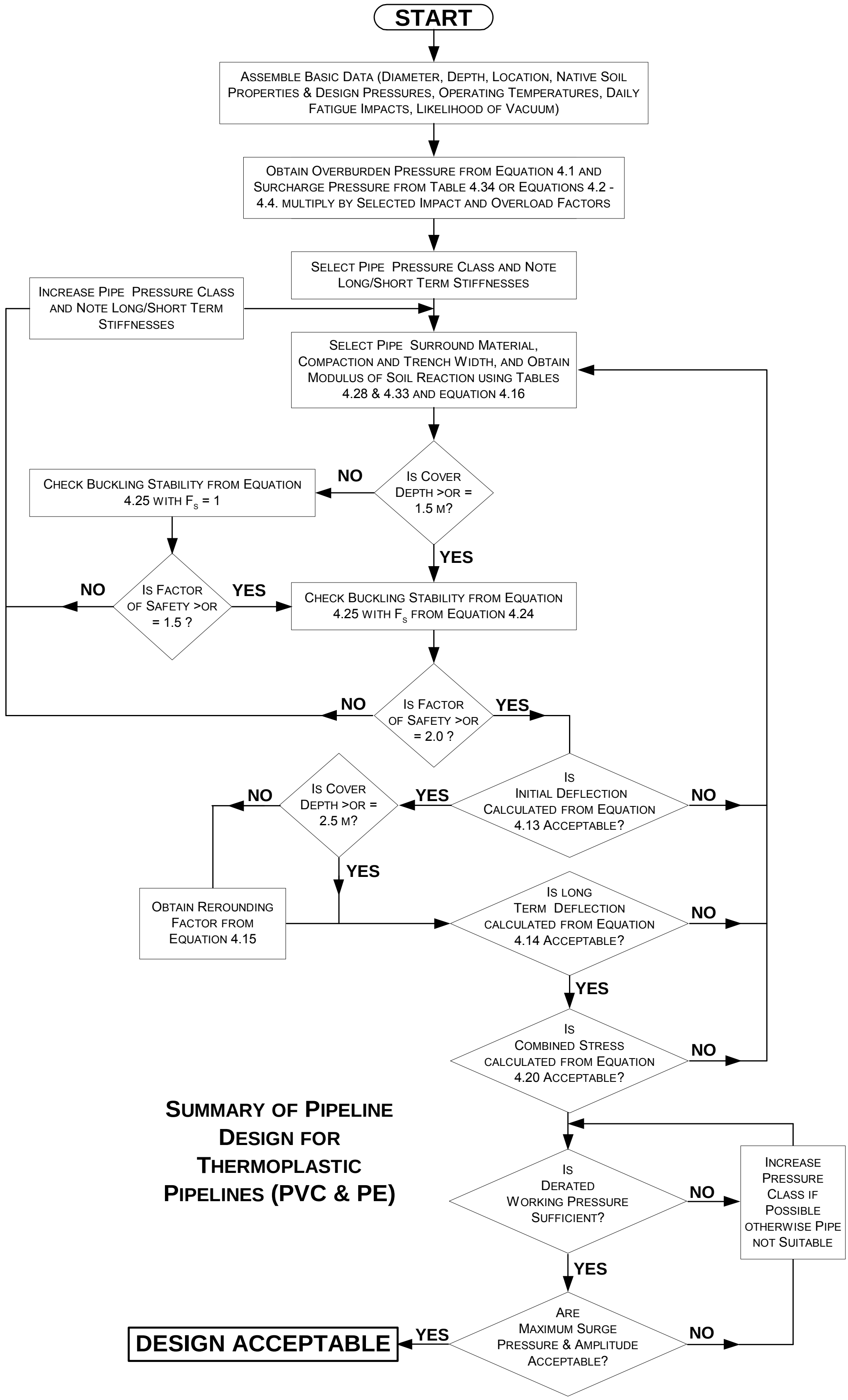
Although many symbols are common to all pipe specifications and standards, there are some notable differences. Those used here are shown below. Where a minimum (min) or maximum (max) value is referred to the additional suffix '$_{min}$' or '$_{max}$' is added:

Description	Symbol	Unit
Compaction of Embedment	80-90	%
Trench width (or width of embanked pipe surround)	B	m
Coefficient for calculation of effective overall soil modulus	C_L	unit less
Celerity (speed) of pressure wave along pipe	c_v	m/s
Celerity (speed) of pressure wave of water	c_w	m/s
Mean diameter of pipe	D	mm
Outside diameter of pipe	DE	mm
Inside diameter of pipe	D_i	mm
Deflection lag factor	D_L	unit less
Nominal diameter	DN	mm
Re-rounding factor	D_R	m
The base of the natural logarithm	e	≈2.718282
Elastic modulus of pipe wall	E	MPa
Effective overall modulus of soil reaction	E'	N/m^2
Modulus of soil reaction for pipe embedment material	E'_2	MPa
Modulus of soil reaction for 'native' ground	E'_3	MPa
Modulus of elasticity of cement mortar lining (ferrous pipes)	E_c	MPa
Short-term elastic steel modulus	E_i	MPa
Long-term elastic steel modulus	E_L	MPa
Bulk modulus of water at operating temperature	E_w	MPa
Combined tensile stress	f_c	MPa
Minimum circular wall stress of PVC @ 20°C & 50 years	f_{circ}	MPa
Hoop tensile stress	f_h	MPa
Factor of Overload	FoL	unit less
Factor of safety (various)	FoS	unit less
Support factor (=1 when cover ≤ 1.5 m)	F_S	unit less
Surge Adjustment Factor (pumping mains 1.0, gravity mains 0.6)	F_{sad}	unit less
Surge Pressure Utilisation Factor	F_{sp}	unit less
Working Pressure Utilisation Factor (FoS=2)	F_{wp}	unit less
Depth of cover to top of pipe	H	mm
Coefficient of active lateral earth pressure	K	unit less
Traffic impact factor, moving vehicle	K_{Tm}	unit less
Traffic impact factor, parked vehicle	K_{Tp}	unit less
Deflection (bedding) coefficient	k_x	unit less

Description	Symbol	Unit
Pipe-soil stiffness factor	n	unit less
Critical external buckling pressure, long = '$_{crl}$', short ='$_{crs}$'	p_{cr}	Bar
PVC de-rating factor	p_{dr}	
Vertical overburden pressure	P_e	N/m^2
Required continuous operating pressure, exclusive of surge	PFA	Bar
Allowable operating pressure	p_i	Bar
Maximum positive pressure increase caused by surge	P_m	Bar
Maximum allowable pressure inclusive of surge	PMA	Bar
Nominal working pressure	PN	Bar
Vacuum load (minimum or negative pressure during surge)	P_n	Bar or N/m^2
Vertical surcharge pressure due to surface loading	P_s	N/m^2
Vertical surcharge pressure due to vehicle	P_v	kN/m^2
Vapour pressure	p_{vp}	Bar
Design discharge	Q	m^3/s
Re-rating Factor for PVC	R_R	unit less
Pipe stiffness (=EI/D^3)	S	N/m^2
Trench embedment class selected	S1 - S4	unit less
Short term pipe stiffness	S_i	N/m^2
Long term pipe stiffness (creep)	S_L	N/m^2
Pipe wall thickness selected from ISO 559 or EN 10224	t	mm
Minus tolerance in pipe wall thickness	t'	mm
Ambient temperature	T_a	^0C
Cement mortar lining mean thickness	t_c	mm
Equivalent pipe wall thickness	t_e	mm
Epoxy lining mean thickness	t_{em}	microns
Operating Water Temperature	T_{op}	^oC
Velocity of flow	V	m/s
Joint factor welding efficiency	V_n	unit less
Soil density	γ_s	N/m^3
Relative deflection at min cover, dependant on lining material	Δ/D	%
Pipe diameter change due to imposed loads	Δ	mm
Design stress	σ_s	MPa
Minimum yield stress for selected steel grade (235, 275, 355)	σ_y	Mpa
Coefficient of friction at trench wall	μ'	unit less

4.16.2 Design of PVCu and PE Pipelines

A procedure that can be adopted in designing PVCu and PE pipelines (WRc 1995) is as follows:



The following example illustrates how the formulae are applied for PVCu (and PEHD) pipelines.

Description	Symbol Used	Unit	Raw Data/Criteria Steel Large Dia. Example	Selected from Table or PVC Calculation using Relevant Formula - small diameter	Remark	Formula Reference in Section 4.16.1
Depth of cover (minimum)	H_{min}	mm	900		Given	
Soil density	γ_s	N/m^3	20000			
Vertical overburden pressure, (minimum soil load)	P_{emin}	N/m^2	-	**18,000**		**(1)**
Traffic impact factor, moving vehicle	K_{Tm}		1.67		**recommended minimum**	
Factor of Overload	F_oL		1.2		**recommended minimum**	
Vertical surcharge pressure due to vehicle	p_v	kN/m^2	48.47		From table	
Vertical surcharge pressure at min depth due to surface loading	P_{smax}	N/m^2	-	**97,134**		**(2)**
Design discharge	Q	m^3/s	0.025		Given	
Approx equivalent Nominal Diameter of D. I. pipe	DN	mm	200		Select	
Outside diameter of pipe	DE	mm	225		From table	
Factor of Safety for diameter selected				**2**		
Req' continuous operating pressure, exclusive of surge	PFA	Bar	6		Given	
Selected pressure, exc.surge (DE <= 90 >= 8 or DE>90 >=10)		Bar	**12.5**	**PASS**		
Pipe wall thickness (determined from Outside Diameter and S)	t	mm		10.6	From table or equation	
Pipe Series	S			**8**		
Minimum circular wall stress @20°C & 50 years		MPa	25			
Maximum design stress	PN	Bar	-	**12.50**		**(3)**
Ambient temperature		°C	30			
Fluid operating temperature	T_{op}	°C	30			
Hydrodynamics - fatigue re-rating (estimated daily occurences0	No.		10			
Hydrodynamics - fatigue re-rating factor				1		
Allow for short term Pressure Fluctuations in Combined Stress Check				**No**		
Vapour pressure	p_{vp}	Bar	0.2			
Vacuum load (-ve surge)	P_n	N/m^2	-	**80,000**		**(5)**
Max. permissible design stress at T_{op} (no hydrodynamic effects)	PMA	Bar		**10.00**		**(6)**
Max. permissible design stress at T_{op} (with hydrodynamic effects)	PMA	Bar		**10.00**		**(6)**
Short term elastic modulus of PVC	E_i	MPa	2,800		Given	**(6)**
Long term elastic modulus of PVC	E_l	MPa	504		Given	**(6)**
Short term pipe stiffness	S_i	N/m^2	-	**28,198**		**(7)**
Long term pipe stiffness (creep)	S_l	N/m^2	-	**5,076**		**(7)**
Modulus of soil reaction for 'native' ground	E'_3	MPa	from Table	**4.50**	For native soil	**(8)**
Modulus of soil reaction for pipe embedment material	E'_2	MPa	from Table	**7.0**	S3	**(9)**
Trench width (or width of embanked pipe surround)	B	m	-	**825**		**(10)**
Coefficient for calculation of effective overall soil modulus	C_L		-	**0.9449**		**(10)**
Effective overall modulus of soil reaction	E'	N/m^2	-	**6,614,016**		**(10)**
Trench embedment class selected (S1,S2,S3,S4)		S3				
Compaction of Embedment (80%, 85%, or 90%)		90				
Deflection lag factor	D_L		from Table	**1.25**	for S3/S4	**(11)**
Deflection coefficient	k_x		from Table	**0.083**	class S3/S4	**(11)**
Rerounding factor	D_R	m	-	**0.6875**		**(11)**
Depth of cover (maximum)	H_{max}	mm	1500			
Traffic impact factor, parked vehicle	K_{Tp}		1.0			
Vertical overburden pressure, (maximum soil load)	P_{emax}	N/m^2	-	**30,000**		**(11)**
Vertical surcharge pressure at max depth due to surface loading	P_{smin}	N/m^2	-	**24,686**		**(11)**
Initial deflection at min cover, dependant on lining material	Δ/D	%	<5% ep : 3% c-m	**2.15**	**PASS**	**(11)**
Long term deflection	Δ/D	%	<5% ep : 3% c-m	**2.24**	**PASS**	**(11)**
Spigot deflection check at max. cover, for steel pipes should be < 2%	Δ/D	%	<2%	**0.80**	**PASS**	**(11)**
Bulk modulus of water at 30 deg C	E_w	MPa	2,230			
Celerity (speed) of pressure wave of water	c_w	m/s	1,425			
Surge Adjustment Factor (pumping mains 1.0, gravity mains 0.6)	F_{sad}		0.5			
Inside diameter of pipe	D_i	mm	-	**203.8**		**(12)**
Velocity of flow	V	m/s	-	**0.77**		**(12)**
Celerity (speed) of pressure wave along pipe	c_v	m/s	-	**352.8**		**(12)**
Maximum short term surge pressure	ΔH	Bar		**1.38**		**(12)**
Maximum positive pressure during surge	P_m	Bar	-	**7.38**	**PASS**	**(12)**
Maximum allowable pressure inclusive of surge	PMA	Bar		**10.00**		
Over surge pressure (amplitude)	P_n	Bar	-	**2.76**		**(12)**
Amplitude within limits (Pass/Fail)				**3.00**	**PASS**	**(12)**
Strain Factor for Pipe Stiffness				**6.5**	**Est. from Table**	
Combined stress for actual working prerssure	f_c	MPa	-	**9.63**	**PASS**	**(13)**
Support factor at minimum depth (no support taken)	F_S		1.00			
Support factor at maximum depth	F_S		-	**5.12**		**(14)**
Critical short term buckling check with minimum cover	p_{crs}	kN/m^2	-	**676.8**		**(14)**
Critical long term buckling check at minimum cover	p_{crl}	kN/m^2	-	**121.8**		**(14)**
FoS against buckling (minimum cover) exc. surcharge	F_oS	minimum	1.5	**3.43**	**PASS**	**(14)**
Derated working pressure	p_{der}			**9.89**	**PASS**	**(15)**

CHAPTER FOUR

A spreadsheet developed to carry out the necessary computations and available from the Design Section of the Ministry of Water upon request is illustrated below. From this it will be noted that for PVC pipes, wall thickness is almost never determined by working pressure alone but has to be increased by one or more pressure classes to satisfy a comprehensive design. The continuing failure to properly design PVC pipelines goes a long way to explain why PVC pipe has gained such a poor reputation in some quarters. -

STRUCTURAL DESIGN OF uPVC PIPES 12.5 No Soil Mod

Design Item	Units	Value	Ref.	Clause
VehicleLoad(Major/Normal/Field/None)		Normal		
Minimum Soil Cover to top of pipe, H	m	0.90		
Maximum Soil Cover to top of pipe, H	m	1.50		
Vertical Wheel Load without factors		48.47		
Vehicle Load Impact Factor (1.5 for good surface, 1.67 for reasonable surface, 2.00 for bad surface)		1.50	A	3.2.3.2
Wheel Load inc overload factor		1.25		
Vertical surcharge from vehical wheel load	kN	90.87	A	3.2.3.2
Vacuum in Pipe ('partial', 'none')		partial		
Hydrodynamics[Surge/Fatigue(Yes/No)] "Yes" if in road reserve, or pumping main,"No" if not		No	C	
Load due to Soil, Pe	N/m^2	18,000	A	
Load due to 'full', 'partial', or 'no' Vacuum, Pn	N/m^2	80,000		
Load due to Vehicle Load, Ps	N/m^2	90,874		
Soil + Vehicle Load, Pe + Ps	N/m^2	108,874		
Soil + Vacuum Load, Pe + Pn	N/m^2	98,000		
Soil + Vehicle + Vacuum Load, Pe + Pn + Ps	N/m^2	188,874		
Ambient Temperature deg C		30		
Fluid Operating Temperature deg C		30		
Fluid Operating Temperature deg K		303		
uPVC Max. Storage Temp BS CP 312 (deg C)		25		
PN Class Selected (bar) (8.0, 10.0, 12.5, 16, 20, 25)		**12.5**	A	3.2.1.1
Actual Internal Working Pressure (bar)		7		
Allow for short term Pressure Fluctuation in Combined Stress Check (No if re-rating factor > 1.0)		Yes		
Estimated Surge Factor for Joukovsky equation (instantaneous = 1)		0.5		
Hydrodynamics - Fatigue re-rating (estimated daily frequency)		10	C	
Hydrodynamic Re-rating factor for estimated daily frequency		1.00	C	
Native Soil (Gravel=G,Sand=S,Clayey Silty Sand=CSS, Clay=C,R=Rock)		CSS		
Condition (Very Dense=VD, Dense=D, Medium Dense=MD, Loose=L, Very Loose=VL)		L		
Native Soil Modulus, E'3 (select appropriate value from table [x 1,000,000])	N/m^2	2,000,000	A	3.2.2.3
Surround Soil Modulus, E'2	N/m^2	7,000,000	A	
Trench Embedment Class (S1,S2,S3)		S3	A	3.3.2.2
Compaction of Surround (80,85,90%)	(% MPD)	90	A	
Deflection Lag Factor, Dl		1.25	A	
Bedding Factor, K		0.0830	A	
Bulk Modulus of Water @ vv ^{o}C	MPa	2,050		
Minimum Buckling Factor of Safety		1.50	A	3.3.5.3
Min. Circ. Wall Stress @20 deg C & 50 years, o.d.<= 90	MPa	25.00	B	
Min. Circ. Wall Stress @20 deg C & 50 years, o.d.> 90	MPa	25.00	B	
Max. Design Stress@20 deg C & 50 years, o.d.<= 90 SF= 2.5	MPa	10.0	B	
Max. Design Stress @20 deg C & 50 years, o.d.> 90 SF= 2.0	MPa	12.5	B	
Max Permissible Design Stress @ Operating Temp, o.d.<= 90 no hydrodynamic effects	MPa	8.00		
Max Permissible Design Stress @ Operating Temp, o.d.> 90	MPa	10.00		
Max Permissible Design Stress @ Operating Temp, o.d.<= 90 with hydrodynamic re-rating factor if applied	MPa	8.00	C	
Max Permissible Design Stress @ Operating Temp, o.d.> 90	MPa	10.00	C	
Elastic Modulus Short Term	MPa	2,800	A	
Elastic Modulus Long Term	MPa	504	A	

Modulus of Soil Reaction: Native Soil, E'3

Soil Condition	Soil Type: Gravel	Sand	Clayey Silty Sand
Medium	12.00	6.50	4.50
Loose	7.00	3.00	2.00
Very Loose	4.00	1.50	1.00

Soil	Soil Type
Very Hard	12.50
Hard	10.50
Very Stiff	8.00
Stiff	5.00
Firm	3.50
Soft	2.25
Very soft	0.75

Note: For diameters > 250 mm, test pit data must be used to determine appropriate soil type, with distance between test pits not greater than 500 m.

Instructions: Complete the items indicated to the left by grey shading to reflect the actual situation for the water supply scheme area where the pipes are to be used

Note: Due to different approaches that can be adopted, it either includes the short term Pressure Fluctuation in the combined stress check OR the long term Hydrodynamic Re-rating factor in the Maximum Permissible Design Stress but not both.

Note: For Joukovsky surge, use a value of 1 for a pumping main where there is no pump flywheel and no surge protection for the pipeline. Otherwise use a factor of 0.75 for pumping mains and 0.5 for distribution mains unless a specific surge calculation is undertaken instead. For pumping mains of diameter > 90 mm, a surge calculation should always be undertaken to check the surge envelope.

Note: For deflection, the check result of the larger of the Soil + Vehicle Load (Pe + Ps) OR Soil + Vacuum Load (Pe + Pn) values is used

Note: This current version of the design calculation does not consider the effects of axial bending due to deflection resulting from ground subsidence risk

Note: Fluid Operating Temperatures will vaty between 30^{o}C at the Coast to 20^{o}C at moderately high elevation inland.

Design method based on WRc Water Pipeline Design Manual 2nd Ed 1995, and WRc uPVC Manual Ed 2002,with appropriate supporting algorithims sourced from elsewhere where the WRc Manuals are silent

A WRC Manual (1995) **B** ISO4422 **C** WRC Manual (2002) Soil Modulus values are average for soil and condition given in the WRc Table 3.4

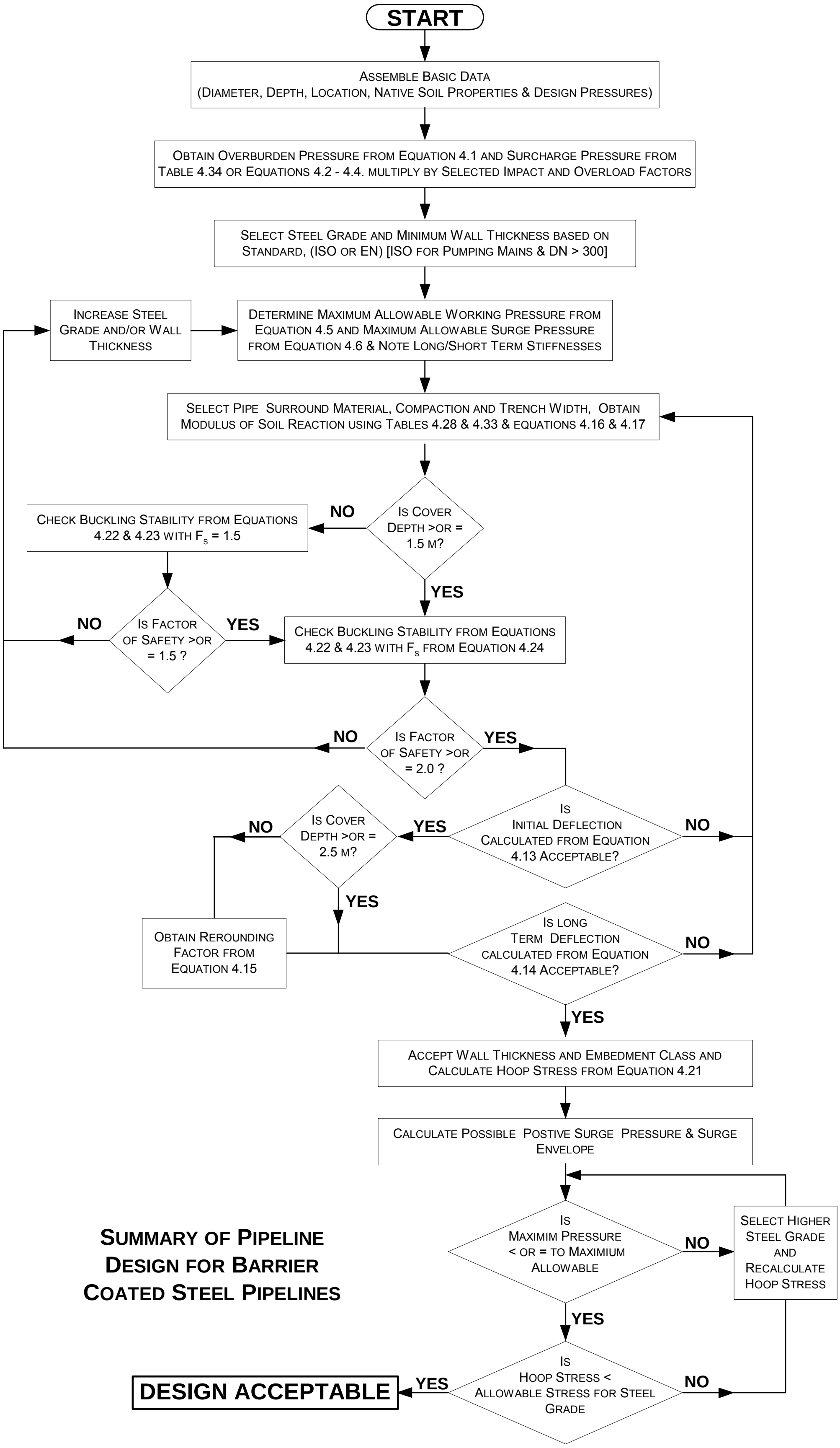
Cont'd

DN	Pipe O.D. (mm)	Short Term Stiffness Si (N/m	Long Term Stiffness (N/m	Long Term Deflec-tion (%)	Strain Factor Df	Com-bined Stress fc (MPa)	Com-bined Stress Check Result	Short Term Support Factor Fsi	Long Term Support Factor FsL	Short Term Critical Pressure pcrs (kN/m	Long Term Critical Pressure pcrL (kN/m	Actual Safety Factor against Buckling	Buckling Check	Overall Pipe Suitability
75	90	57,168	10,290	1.8%	4.6	9.76	Fail!	1.00	1.00	1372.03	247.0	5.07	Pass	FAIL
100	110	28,504	5,131	2.0%	5.5	9.85	Pass	1.00	1.00	684.10	123.1	2.53	Pass	PASS
100	125	29,908	5,383	2.0%	5.4	9.73	Pass	1.00	1.00	717.79	129.2	2.65	Pass	PASS
125	140	29,629	5,333	2.0%	5.4	9.76	Pass	1.00	1.00	711.10	128.0	2.63	Pass	PASS
150	180	29,474	5,305	2.0%	5.4	9.78	Pass	1.00	1.00	707.38	127.3	2.61	Pass	PASS
200	225	29,044	5,228	2.5%	5.5	10.42	Fail!	1.00	1.00	697.06	125.5	2.57	Pass	FAIL
225	250	29,130	5,243	2.7%	5.5	10.68	Fail!	1.00	1.00	699.12	125.8	2.58	Pass	FAIL
250	280	28,938	5,209	2.9%	5.5	10.98	Fail!	1.00	1.00	694.51	125.0	2.56	Pass	FAIL
300	355	29,141	5,245	3.3%	5.5	11.53	Fail!	1.00	1.00	699.38	125.9	2.58	Pass	FAIL

DN	Pipe O.D. (mm)	Min. Wall thick-ness (mm)	Internal Bore (mm)	Derated Working Pressure (bars)	Is Derated Working Pressure sufficient for Actual Int. Pressure?	Flow Velocity (m/s)	Short Term Joukovsky Rise 0.5 (bars)	Max. Pressure Under Surge Pmax (bars)	Min. Pressure Under Surge Pmin (bars)	Are Surge Pressure within Allowed Surge Envelope	Long Term Joukovsky Pressure Rise x 0.5 (bars)	Trench Width Min Side Clearance= 300 (mm)	Overall Soil Modulus Coef-ficient CL	Overall Soil Modulus E' (N/m
75	90	5.3	79.4	10.0	Yes	0.8	1.7	8.7	5.3	Yes	0.7	690.0	1.0	7,000,000
100	110	5.2	99.6	9.9	Yes	0.8	1.5	8.5	5.5	Yes	0.7	710.0	1.0	7,000,000
100	125	6.0	113.0	10.1	Yes	0.8	1.5	8.5	5.5	Yes	0.7	725.0	1.0	7,000,000
125	140	6.7	126.6	10.1	Yes	0.8	1.5	8.5	5.5	Yes	0.7	740.0	1.0	7,000,000
150	180	8.6	162.8	10.0	Yes	1.3	2.4	9.4	4.6	Yes	1.0	780.0	1.0	6,953,192
200	225	10.7	203.6	10.0	Yes	1.3	2.4	9.4	4.6	Yes	1.0	825.0	0.8	5,544,057
225	250	11.9	226.2	10.0	Yes	1.3	2.4	9.4	4.6	Yes	1.0	850.0	0.7	5,060,366
250	280	13.3	253.4	10.0	Yes	1.6	3.0	10.0	4.0	No!	1.3	880.0	0.7	4,629,687
300	355	16.9	321.2	10.0	Yes	1.6	3.0	10.0	4.0	No!	1.3	955.0	0.6	3,945,295

4.16.2 Design of Steel Pipelines

A procedure that can be adopted in designing Steel pipelines (WRc 1995) is as follows:



SUMMARY OF PIPELINE DESIGN FOR BARRIER COATED STEEL PIPELINES

The following examples illustrate how the formulae are applied for Steel pipelines.

Example 2. Large diameter (ND 700 pipe) in Pumping Main with PFA of 20 Bar, steel thickness selected from ISO 559, minimum yield stress of steel 275 MPa.

Description	Symbol Used	Unit	Raw Data/Criteria Steel Large Dia. Example	Selected from Table or Steel Calculation using Relevant Formula - large diameter	Remark	Formula Reference in Section 4.16.1
Depth of cover (minimum)	H_{min}	mm	900		Given	
Soil density	γ_s	N/m^3	20000			
Vertical overburden pressure, (minimum soil load)	P_{emin}	N/m^2	-	**18,000**		**(1)**
Traffic impact factor, moving vehicle	K_{Tm}		1.67		**recommended**	
Factor of Overload	F_oL		1.2		**recommended**	
Vertical surcharge pressure due to vehicle	p_v	kN/m^2	73		From table	
Vertical surcharge pressure at min depth due to surface loading	P_{smax}	N/m^2	-	**97,528**		**(2)**
Design discharge	Q	m^3/s	0.5		Given	
Nominal diameter	DN	mm	700		Select	
Outside diameter of pipe	DE	mm	711		From table	
Working Pressure Utilisation Factor (Factor of Safety=2)	F_{wp}		0.5		AWWA M11	
Required continuous operating pressure, exclusive of surge	PFA	Bar	20		Given	
Minimum operating pressure for vacuum effect calculation		Bar	10.0		estimated	
Pipe wall thickness selected from ISO 559 or EN 10224	t	mm	7.1		ISO 559	
Minus tolerance in pipe wall thickness	t'	mm	0.35		ISO 559	
Joint factor welding efficiency	V_n		1		BSCP2010 Pt2	
Minimum yield stress for selected steel grade (235,275,355)	σ_y	Mpa	235	**PASS**	Select	
Nominal pressure (max allowable)	PN	Bar	-	**23.47**		**(3)**
Surge Pressure Utilisation Factor	F_{sp}		0.75		AWWA M11	
Maximum allowable pressure inclusive of surge	PMA	Bar	-	**35.20**		**(4)**
Vapour pressure	p_{vp}	Bar	0.2			
Vacuum load (-ve surge)	P_n	N/m^2	-	**80,000**		**(5)**
Short term elastic modulus of steel	E_i	MPa	207,000		Select	**(6)**
Long term elastic modulus of steel	E_l	MPa	207,000		Select	**(6)**
Modulus of elasticity of cement mortar	E_c	MPa	28,000			
Cement mortar lining mean thickness (=0 if epoxy lining selected)	t_{cm}	mm	0		Given or table	
Short term pipe stiffness	S_i	N/m^2	-	**15,189**		**(7)**
Long term pipe stiffness (creep)	S_l	N/m^2	-	**15,189**		**(7)**
Modulus of soil reaction for 'native' ground	E'_3	MPa	from Table	**2.50**	For native soil	**(8)**
Trench embedment class selected (S1,S2,S3,S4)		S4				
Compaction of Embedment (80%, 85%, or 90%)		90				
Deflection lag factor	D_L		from Table	**1.25**	for S3/S4	**(9)**
Deflection coefficient	k_x		from Table	**0.100**	class S3/S4	**(9)**
Modulus of soil reaction for pipe embedment material	E'_2	MPa	from Table	**5.0**	S4 as < S3	**(10)**
Trench width (or width of embanked pipe surround)	B	m	-	**1311**		**(11)**
Coefficient for calculation of effective overall soil modulus	C_L		-	**0.6347**		**(11)**
Effective overall modulus of soil reaction	E'	N/m^2	-	**3,173,540**		**(11)**
Depth of cover (maximum)	H_{max}	mm	1500			
Rerounding factor	D_R	m	-	**0.5**		**(12)**
Epoxy lining mean thickness	t_{em}	microns	350			
Traffic impact factor, parked vehicle	K_{Tp}		1.0			
Vertical overburden pressure, (maximum soil load)	P_{emax}	N/m^2	-	**30,000**		**(12)**
Vertical surcharge pressure at max depth due to surface loading	P_{smin}	N/m^2	-	**37,179**		**(12)**
Crown deflection at min cover, dependant on ling material	Δ/D	%	<5% ep : 3% c-m	**3.81**	**PASS**	**(12)**
Spigot deflection check at max cover, for steel pipes must be < 2%	Δ/D	%	<2%	**1.19**	**PASS**	**(12)**
Bulk modulus of water at 30 deg C	E_w	MPa	2,230			
Celerity (speed) of pressure wave of water	c_w	m/s	1,425			
Surge Adjustment Factor (pumping mains 1.0, gravity mains 0.6)	F_{sad}		1.0			
Velocity of flow	V	m/s	-	**1.3**		**(13)**
Inside diameter of pipe	D_i	mm	-	**696.1**		**(13)**
Celerity (speed) of pressure wave along pipe	c_v	m/s	-	**832.5**		**(13)**
Maximum surge pressure	ΔH	Bar		**11.15**		**(13)**
Maximum positive pressure during surge	P_m	Bar	-	**31.15**	**PASS**	**(13)**
Maximum allowable pressure inclusive of surge	PMA	Bar		**35.20**		
Minimum negative pressure during surge	P_n	Bar	-	**-1.15**		**(13)**
Allow for risk of vacuum in pipeline					**YES**	
Hoop tensile stress for actual required working pressure	f_h	MPa	-	**98**	**PASS**	**(14)**
Design stress	σ_s	MPa	-	**117.5**		**(14)**
Support factor at minimum depth (no support taken)	F_S		1.00			
Support factor at maximum depth	F_S		-	**4.27**		**(15)**
Critical short term buckling check with minimum cover	p_{crs}	kN/m^2	-	**364.5**		**(15)**
Critical long term buckling check	p_{crl}	kN/m^2	-	**1558.0**		**(15)**
Factor of safety against buckling (minimum cover) exc. surcharge	F_oS	minimum	1.5	**3.72**	**PASS**	**(15)**
Factor of safety against buckling (maximum depth) inc. surcharge	F_oS	minimum	1.5	**7.51**	**PASS**	**(15)**

The surge calculation made here is approximate only, to be used as a guide. For pumping mains of DN> 200 this should be checked by a detailed surge analysis taking into account any surge protection devices included in the system.

Example 3. Small diameter (ND 200 pipe) in Distribution with PFA of 6 Bar, steel thickness selected from EN 10224, minimum yield stress of steel found necessary: 275 MPa.

Description	Symbol Used	Unit	Raw Data/Criteria Steel Small Dia. Example	Selected from Table or Steel Calculation using Relevant Formula - small diameter	Remark	Formula Reference in Section 4.16.1
Depth of cover (minimum)	H_{min}	mm	600		Given	
Soil density	γ_s	N/m^3	20000			
Vertical overburden pressure, (minimum soil load)	P_{emin}	N/m^2	-	**12,000**		**(1)**
Traffic impact factor, moving vehicle	K_{Tm}		1.67		**recommended**	
Factor of Overload	F_oL		1.2		**recommended**	
Vertical surcharge pressure due to vehicle	p_v	kN/m^2	120		From table	
Vertical surcharge pressure at min depth due to surface loading	P_{smax}	N/m^2	-	**160,320**		**(2)**
Design discharge	Q	m^3/s	0.03		Given	
Nominal diameter	DN	mm	200		Select	
Outside diameter of pipe	DE	mm	219.1		From table	
Working Pressure Utilisation Factor (Factor of Safety=2)	F_{wp}		0.5		AWWA M11	
Required continuous operating pressure, exclusive of surge	PFA	Bar	6		Given	
Minimum operating pressure for vacuum effect calculation		Bar	2		estimated	
Pipe wall thickness selected from ISO 559 or EN 10224	t	mm	2		EN 10224	
Minus tolerance in pipe wall thickness	t'	mm	0.25		EN 10224	
Joint factor welding efficiency	V_n		1		BSCP2010 Pt2	
Minimum yield stress for selected steel grade (235,275,355)	σ_y	Mpa	275	**PASS**	Select	
Nominal pressure (max allowable)	PN	Bar	-	**7.53**		**(3)**
Surge Pressure Utilisation Factor	F_{sp}		0.75		AWWA M11	
Maximum allowable pressure inclusive of surge	PMA	Bar	-	**11.30**		**(4)**
Vapour pressure	p_{vp}	Bar	0.2			
Vacuum load (-ve surge)	P_n	N/m^2	-	**80,000**		**(5)**
Short term elastic modulus of steel	E_i	MPa	207,000		Select	**(6)**
Long term elastic modulus of steel	E_l	MPa	207,000			**(6)**
Modulus of elasticity of cement mortar	E_c	MPa	28,000			
Cement mortar lining mean thickness (=0 if epoxy lining selected)	t_{cm}	mm	4		Given or table	
Short term pipe stiffness	S_i	N/m^2	-	**24,381**		**(7)**
Long term pipe stiffness (creep)	S_l	N/m^2	-	**24,381**		**(7)**
Modulus of soil reaction for 'native' ground	E'_3	MPa	from Table	**2.50**	For native soil	**(8)**
Trench embedment class selected (S1,S2,S3,S4)		S4				
Compaction of Embedment (80%, 85%, or 90%)		90				
Deflection lag factor	D_L		from Table	**1.25**	for S3/S4	**(9)**
Deflection coefficient	k_x		from Table	**0.100**	class S3/S4	**(9)**
Modulus of soil reaction for pipe embedment material	E'_2	MPa	from Table	**5.0**	S4 as < S3	**(10)**
Trench width (or width of embanked pipe surround)	B	m	-	**819.1**		**(11)**
Coefficient for calculation of effective overall soil modulus	C_L		-	**0.9150**		**(11)**
Effective overall modulus of soil reaction	E'	N/m^2	-	**4,575,240**		**(11)**
Depth of cover (maximum)	H_{max}	mm	1500			
Rerounding factor	D_R	m	-	**0.85**		**(12)**
Epoxy lining mean thickness	t_{em}	microns	350			
Traffic impact factor, parked vehicle	K_{Tp}		1.0			
Vertical overburden pressure, (maximum soil load)	P_{emax}	N/m^2	-	**30,000**		**(12)**
Vertical surcharge pressure at max depth due to surface loading	P_{smin}	N/m^2	-	**61,115**		**(12)**
Crown deflection at min cover, dependant on ling material	Δ/D	%	<5% ep : 3% c-m	**3.70**	**PASS**	**(12)**
Spigot deflection check at max cover, for steel pipes must be < 2%	Δ/D	%	<2%	**1.77**	**PASS**	**(12)**
Bulk modulus of water at 30 deg C	E_w	MPa	2,230			
Celerity (speed) of pressure wave of water	c_w	m/s	1,425			
Surge Adjustment Factor (pumping mains 1.0, gravity mains 0.6)	F_{sad}		0.6			
Velocity of flow	V	m/s	-	**0.9**		**(13)**
Inside diameter of pipe	D_i	mm	-	**206.4**		**(13)**
Celerity (speed) of pressure wave along pipe	c_v	m/s	-	**821.5**		**(13)**
Maximum surge pressure	ΔH	Bar		**4.50**		**(13)**
Maximum positive pressure during surge	P_m	Bar	-	**10.50**	**PASS**	**(13)**
Maximum allowable pressure inclusive of surge	PMA	Bar		**11.30**		
Minimum negative pressure during surge	P_n	Bar	-	**-2.50**		**(13)**
Allow for vacuum in pipeline					**YES**	
Hoop tensile stress for actual required working pressure	f_h	MPa	-	**32**	**PASS**	**(14)**
Design stress	σ_s	MPa	-	**137.5**		**(14)**
Support factor at minimum depth	F_S		1.00			
Support factor at maximum depth	F_S		-	**6.55**		**(15)**
Critical short term buckling check with minimum cover	p_{crs}	kN/m^2	-	**585.1**		**(15)**
Critical long term buckling check	p_{crl}	kN/m^2	-	**3834.4**		**(15)**
Factor of safety against buckling (minimum cover) exc. surcharge	F_oS	minimum	1.5	**6.36**	**PASS**	**(15)**
Factor of safety against buckling (maximum depth) inc. surcharge	F_oS	minimum	1.5	**14.18**	**PASS**	**(15)**

A spreadsheet developed to carry out the necessary computations other than for surge is illustrated below. This is available from the local spiral-weld steel pipe manufacturer upon request.

DESIGN OF SPIRAL WELDED STEEL PIPES WITH HOT-ROLLED STEEL BASED ON MINIMUM WALL THICKNESSES TO ISO 559

Minimum Soil Cover, H		1. =	0.9
Trench Embedment Class (S1,S2,S3,S4)		2. =	S4
Compaction of Embedment (80,85,90%	(MPD)	3. =	90
Modulus of Soil Reaction, E'2	N/m2		5,000,000
Trench Bedding Support Angle	(deg)		60
Embedment Deflection Lag Factor, DL			1.25
Bedding Factor, K			0.103
Minimum Buckling Factor of Safety			1.5
Steel Min. Yield/Proof Stress	MPa	4. =	275.0
Steel Design Stress, fa <=200	MPa		137.5
Steel Design Stress, fa >200	MPa		137.5
Steel Elastic Modulus Short Term	MPa		207,000
Steel Elastic Modulus Long Term	MPa		207,000

The Designer must select the following:

1. Minimum soil cover which must be 0.9 m or greater beside Major Roads. Elsewhere, it must be 0.6 m or greater

If in doubt select S4 as the minimum requirement.

3. The compaction achievable. If in doubt select 80% (S1 to S3) or 85% (S4). However, 90% is acheivable in all cases when care is taken.

If grade 355 insufficient consult manufacturer for alternatives]

5. The possible Vehicle Load Type [Note: 'Major Road' only applies to major trunk roads. In most cases 'Normal Road' is sufficient]

7. Vehicle Overload Factor (1.2 recommended) 8. Risk of Vacuum in Pipe (Yes/No)

9. Minimum clearance on each side of pipe to allow for proper compaction of bedding. (300 mm recommended) 10. Native soil modulus.

Vehicle Load Type (Major Road/Normal Road/Field /None)	5. =	Normal Road
Vehicle Load Impact Factor	6. =	1.67
Vehicle Overload Factor	7. =	1.20
Risk of Vacuum in Pipe (Yes/No)	8. =	Yes
Trench width = 1.5 x OD subject to minimum clearance of:	9. =	300
98.4		

Native soil modulus, E'_{3}	N/m2	2,500,000
Load due to Soil, Pe	N/m2	18,000
Load due to Vacuum (water vapour), Pn	N/m2	80,000
Load due to Vehicle Load, Ps	N/m2	97,126
Soil + Vehicle Load, Pe + Ps	N/m2	115,126
Soil + Vacuum Load, Pe + Pn	N/m2	98,000

Steel Pipe EN10224

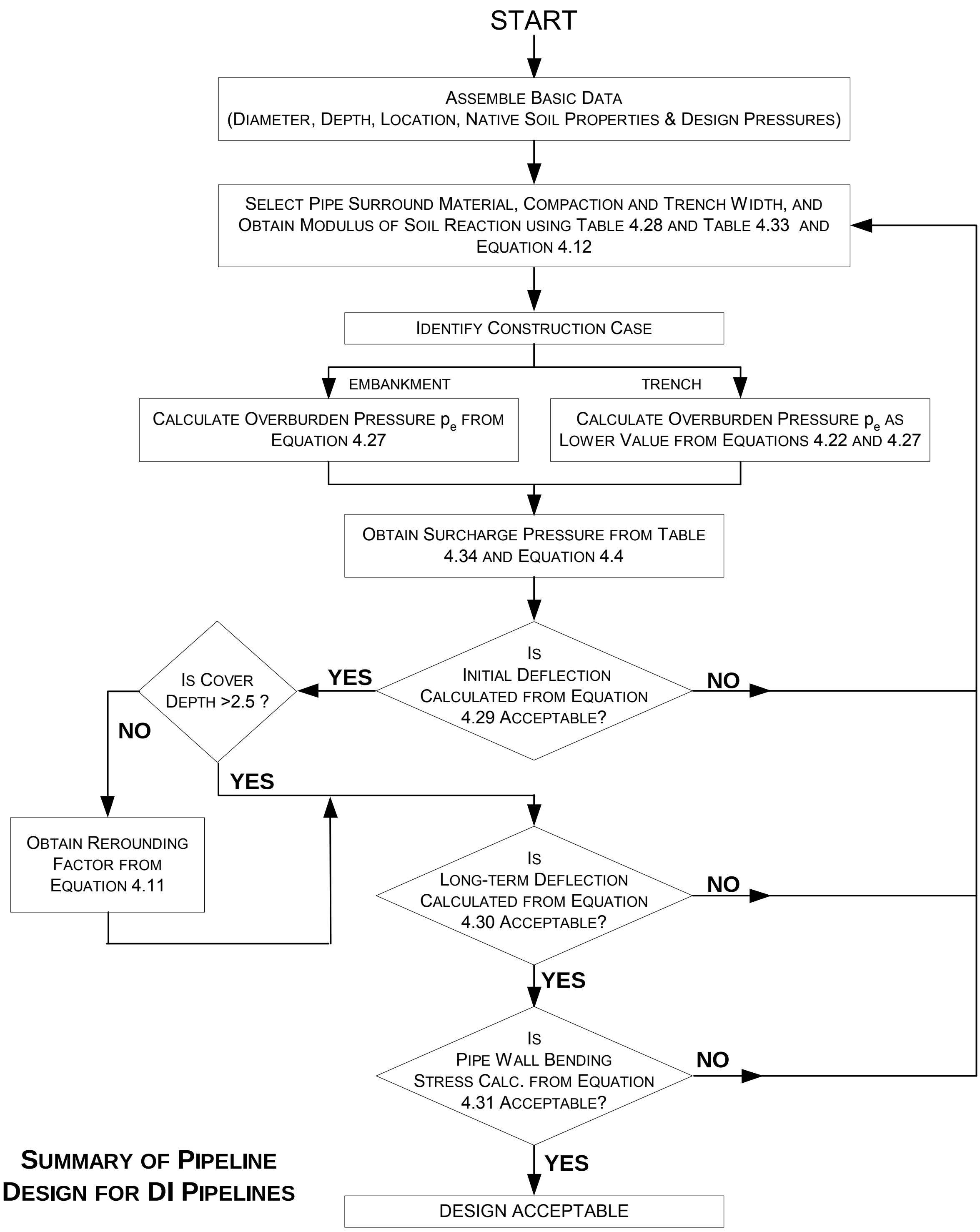
Weld Efficency,Vn= 1.0 F.O.S for Int. Pressure , Y=

DN	Pipe O.D. (mm)	Max. Pipe O.D. (mm)	Nom. steel thick-ness (mm)	Wall Thick-ness Tols. (mm)	Nom. cement thick-ness (mm)	Min. cement thick-ness (mm)	Nom. bore (mm)	Max. bore (mm)	Max. Static Working Pressure (bars)	Nom. Pres-sure PN (bars)	Lining (cement mortar) or (epoxy)	Long Term Stiffness (N/m2)	Trench Width (mm)	Effective Overall Soil Modulus (N/m2)	Short Term Deflec-tion (%)	Crown Deflection Check Result (Pass/Fail)	Hoop Tensile Stress fc (MPa)	Hoop Tensile Stress Check (Pass/Fail)	Support Factor Fs	Critical Pressure pcr (N/m2)	Actual Safety Factor against Buckling	Buckling Check (Pass/Fail)
65	76.1	77	2.3	0.25	4.0	3.0	63.5	66.9	86.7	80.0	Cement	805123	676	5,000,000	**0.2%**	**Pass**	128	**Pass**	1.00	19,322,943	197.2	**Pass**
80	88.9	90	2.3	0.25	4.0	3.0	76.3	79.9	70.5	63.0	Cement	491833	689	5,000,000	**0.3%**	**Pass**	118	**Pass**	1.00	11,804,002	120.4	**Pass**
100	114.3	115	2.6	0.25	4.0	3.0	101.1	104.3	64.6	63.0	Cement	279098	714	5,000,000	**0.5%**	**Pass**	135	**Pass**	1.00	6,698,344	68.4	**Pass**
150	168.3	170	2.6	0.25	4.0	3.0	155.1	159.3	43.1	40.0	Cement	84305	768	5,000,000	**1.3%**	**Pass**	127	**Pass**	1.00	2,023,326	20.6	**Pass**
200	219.1	221	2.6	0.25	4.0	3.0	205.9	210.3	32.3	32.0	Cement	37547	819	4,575,240	**2.1%**	**Pass**	132	**Pass**	1.00	901,127	9.2	**Pass**
250	273	276	3.6	0.35	5.0	3.0	255.8	263.5	35.8	32.0	Cement	45948	873	4,189,886	**2.0%**	**Pass**	119	**Pass**	1.00	1,102,762	11.3	**Pass**
300	323.9	327	4.0	0.35	5.0	3.5	305.9	312.7	33.5	32.0	Cement	34879	924	3,937,765	**2.4%**	**Pass**	127	**Pass**	1.00	837,102	8.5	**Pass**
350	355.6	359	4.0	0.35	5.0	3.5	337.6	344.7	28.7	25.0	Cement	26246	956	3,815,530	**2.8%**	**Pass**	109	**Pass**	1.00	629,898	6.4	**Pass**
400	406.4	410	4.5	0.35	6.0	3.5	385.4	394.7	28.5	25.0	Cement	27065	1006	3,657,790	**2.8%**	**Pass**	111	**Pass**	1.00	649,561	6.6	**Pass**
450	457.2	462	5.0	0.35	6.0	4.5	435.2	443.7	28.4	25.0	Cement	24387	1057	3,533,829	**3.0%**	**Fail**	112	**Pass**	1.00	585,278	6.0	**Pass**
500	508	513	5.0	0.35	6.0	4.5	486.0	494.7	25.6	25.0	Cement	17705	1108	3,433,844	**3.5%**	**Fail**	125	**Pass**	1.00	424,930	4.3	**Pass**
600	610	616	5.6	0.35	0.0	6.0	598.8	593.5	24.1	25.0	Epoxy	11286	1210	3,281,991	**4.2%**	**Pass**	134	**Pass**	1.00	270,864	2.8	**Pass**
700	711	718	7.1	0.35	0.0	6.0	696.8	692.5	26.5	25.0	Epoxy	15189	1311	3,173,540	**3.9%**	**Pass**	123	**Pass**	1.00	364,528	3.7	**Pass**
800	813	821	8.0	0.35	0.0	8.0	797.0	789.7	26.3	25.0	Epoxy	14785	1413	3,090,794	**4.0%**	**Pass**	125	**Pass**	1.00	354,838	3.6	**Pass**
900	914	923	8.8	0.35	0.0	8.0	896.4	890.1	25.8	25.0	Epoxy	14016	1514	3,026,720	**4.2%**	**Pass**	128	**Pass**	1.00	336,382	3.4	**Pass**
1000	1016	1026	10.0	0.35	0.0	10.0	996.0	986.7	26.5	25.0	Epoxy	15210	1616	2,974,724	**4.1%**	**Pass**	125	**Pass**	1.00	365,036	3.7	**Pass**
1200	1220	1232	11.0	0.50	0.0	12.0	1198.0	1187.0	24.1	25.0	Epoxy	11286	1830	2,902,966	**4.6%**	**Pass**	136	**Pass**	1.00	270,864	2.8	**Pass**
1400	1420	1434	13.0	0.50	0.0	12.0	1394.0	1385.0	24.6	25.0	Epoxy	12083	2130	2,902,966	**4.5%**	**Pass**	134	**Pass**	1.00	289,991	3.0	**Pass**
1600	1620	1636	16.0	0.50	0.0	12.0	1588.0	1581.0	26.6	25.0	Epoxy	15551	2430	2,902,966	**4.1%**	**Pass**	124	**Pass**	1.00	373,230	3.8	**Pass**
1800	1820	1838	17.5	0.50	0.0	12.0	1785.0	1780.0	26.0	25.0	Epoxy	14459	2730	2,902,966	**4.2%**	**Pass**	128	**Pass**	1.00	347,024	3.5	**Pass**
2000	2020	2040	20.0	0.50	0.0	12.0	1980.0	1977.0	26.8	25.0	Epoxy	15976	3030	2,902,966	**4.0%**	**Pass**	124	**Pass**	1.00	383,432	3.9	**Pass**

Note:

4.16.3 Design of D.I. Pipelines

Although rarely necessary, a procedure that can be adopted to design a D.I. pipeline (WRc 1995) is as follows:



SUMMARY OF PIPELINE DESIGN FOR DI PIPELINES

Because of the rarity of needing to carry out this calculation, no example is presented.

4.17 PIPE FITTINGS AND VALVES

4.17.1 Pipe Fittings

Pipe fittings are those specially manufactured pipe pieces used to facilitate changes in direction, changes in diameter, the making of branches etc. to the pipeline.

Further fittings are needed to install valves, meters and other mechanical devices and to allow for the change from one pipe material to another.

Consideration should also be given to the type of joints of the selected pipe materials, i.e. whether rigid or flexible. Spigot and socket joints are flexible allowing between 3° to 6° deviation at each joint. Other flexible joints are tyton joints, tyton anchor joints, tyton lock joints, flexible / mechanical couplings, grooved couplings and screwed gland joints. Rigid joints include glued joints (thermoplastic pipes), grooved joints, welded joints:, flanged joints and screw socket joints. However, glued joints are not to be used.

Expansion joints are only needed for pipes with rigid joints especially above ground. Rigid joints are normally thrust resistant and separate anchorage only needs consideration for large diameter, high pressure gusseted steel tees and bends if this is not catered for by the manufacturer. A guide to thrust block design is presented in Section 4.18.xx.

Anchorage is necessary for all flexibly jointed pipes and simple dismantling joints to resist the tendency of pipes to pull apart at bends or other points of unbalanced pressures, or when laid on steep gradients. Anchors are also used to restrain the expansion and contraction of rigidly jointed pipes above ground and under the influence of temperature changes.

Anchorage takes the form of mass concrete blocks at all horizontal and vertical changes of directions, at tees and Y-pieces and on steep gradients. Stop valves also should be anchored by concrete block at right angles to the flow.

The sizes of the concrete anchorages and thrust blocks can be calculated according to the pressure in the pipe, the pipe diameter and the bearing capacity of the ground against which they bear.

Anchor blocks should be so constructed that slightly less than half of the outside diameter of the pipe or fitting is in contact with the concrete, whilst the joints should be left free.

Many different specials for connecting polyethylene pipes to pipes of other materials are available, including tees, and saddles. Polythene pipes can also be joined by electro-fusion or heat welding for which simple electric welding apparatus are on the market. The Designer should consult the manufacturer’s catalogues for the selection of the fittings and specials required. It is inadvisable to use PVC tees and PVC bends should not be used as in their formation, wall thickness reduces.

It is up to the Designer to ascertain the availability of certain type of specials and also to choose the most economical solution. In some instances it might prove more economical to change from one type of pipe material to another to allow for the use of a specific fitting or valve.

In general frictional head losses in pipe specials are negligible from a Designers point of view, except at pumping stations, on treatment works sites and at tank sites. When considered necessary the method of including for such losses recommended here is to calculate equivalent lengths of straight pipeline in accordance with the information given in Table 4.33.

Where there is space for laying horizontally in big radius curves, small directional changes, should be by the use of the few degrees of flexibility available in each of the flexible pipe joints and standard bends should be avoided. This will be more economical in cost and in the flow characteristics. However, where this is done it is important that pipe marker posts clearly demarcate this. In profile, pipes should be laid in straight lines between specific points of change with trench depth varying to accommodate small variations in ground level.

When anchoring pipes, steel straps anchored into the concrete and consisting of two halves bolted together around the pipe can be employed in many cases. In general, anchorages should be designed so that the pipes and specials can be removed and replaced. Only in special cases can pipes or fitting be totally encased in concrete and where necessary it should be done in such a manner that the joints are kept free.

PVC pipes should never fully be encased in concrete and actual contact between the PVC pipe and the concrete should be prevented by using a layer of bitumen paper or polythene film between pipe and concrete.

Break-pressure tanks should be provided wherever the static head become higher than allowable for the pipeline. The inlet pipe should be fitted with ball valve and adequate provision should be made at the overflow to prevent erosion.

4.17.2 Valves

Because of their importance in any pipeline, the specifying of valves is extremely important and the following is given as a guide in this respect.

In general, isolating valves up to and including DN 300 should be gate valves, and valves larger than DN 300 should be butterfly valves except where otherwise specifically required.

All valves should bear an identification mark on the upper body that includes:-

- the name of the manufacturer and/or his trade mark
- the nominal diameter (DN)
- the nominal pressure (PN)

Manufacturer's full technical leaflets should be supplied for approval prior to confirmation of any order for valves.

All valves should be designed for a maximum permissible differential working pressure of 16 Bar except where higher pressures are necessary. All valves should close when the stem rotation is in a clockwise direction unless otherwise specified.

Gate valves up to and including DN 300 should be made of epoxy coated cast ductile iron in accordance with BS 5163-1:2004 unless otherwise specified. The gate should be completely rubber encapsulated, the gate valve being of the pocketless type with a straight through port.

They should be provided with integral flanged ends unless otherwise specified, and the face to face dimensions of gate valves with integral flanged ends should be in accordance with ISO 5752 basic series 14 (short) or basic series 15(long).

They should be of the non-rising stem type except where specifically required otherwise. The stem seal shall be of toroidal sealing rings (O-rings) with at least two such seals. Seals shall be capable of being replaced with the valve under pressure and in the fully open position.

The gate should be of ductile iron fully rubber encapsulated, the gate sealing in the body being ensured by compressing of the rubber. Wedge/gate guides of wear resistant plastic with high gliding features should be provided in the body, optimally placed to guarantee low wear and tear of the gate and low closing torques.

The bonnet gasket should be of elastomer (suitable for potable water). The bonnet studs or allen screws should be corrosion-protected and in addition the studs/allen screws should be fitted in countersunk holes in the bonnet and completely sealed with wax or other suitable material, which can be removed by low-temperature melting in case they have to be disassembled.

Prior to coating, the gate valves shall be works cleaned and shot-blasted in accordance with BS 7079:1989 (C1 to C4) and immediately thereafter, should be coated internally and externally with fusion bonded powder epoxy or equivalent suitable for potable water and to a minimum thickness of 150 microns. For aggressive soil conditions this should be doubled to 300 microns thickness. The body, the bonnet and the gate of the valve should be made of ductile iron to BS 1564:1997, the gate being encapsulated with elastomer EPDM, nitrile or equivalent.

The operating stem should be made of stainless steel at least equivalent to EN 10272:2000, except for use in areas of aggressive soils where this should be to DIN 17440:14404.

The stem nut shall normally be made of high tensile brass, except for use in areas of aggressive soils where this should be aluminium bronze.

Furthermore and in aggressive soils, outside bolts and nuts shall be made of stainless steel to DIN 17440:14301.

Butterfly valves for manual operation should comply with EN 593:2004 and should be double flanged, resilient and metal seated tight shut-off design and of the eccentric disc type supported from two shafts placed in self lubricating bearing bushes.

They should be capable of sustaining a maximum permissible differential working pressure of not less than 16 Bar except where otherwise required.

They should operate with a clockwise closing direction. The valve disk should rotate though an angle between 0 degrees and 90 inclusive and when fully closed should not be capable of being over-tightened. The sealing ring should be made of EPDM rubber and should be attached at the disk edge circumference by a retaining ring without adjustment to form a resilient and durable seal.

The valve disc seal should be replaceable without dismantling the operating mechanism, disk or shafts, and without removing the valve from the pipeline.

The valve should be equipped with an irreversible and proportional worm gear operator. This should be either with or without an additional primary reduction gear placed within a waterproof housing dependant upon nominal valve diameter and maximum working conditions.

The operating mechanism should be permanently lubricated, not in contact with the water and fitted with an OPENED/CLOSED proportional position indicator in order to indicate the disk angular position. The mechanism should be sized in order to minimise torque for ease of manual operation under maximum differential pressure and should be with high class enclosure IP67 to EN 60529:1992. Valve body, disk and disk retaining ring should be in ductile "SG" iron casting to EN 1564:1997. Disk shafts should be in stainless steel to EN 10272:2000.

The valve body should be cleaned and shot blasted to BS 7079:1989 (C1 to C4) and internally and externally protected with a paint suitable for potable water.

Each butterfly valve should be works pressure tested in accordance with ISO 5208:1993.

- body test at a minimum pressure of 1.5 times the maximum permissible pressure
- seal test at a minimum pressure of 1.1. times the maximum permissible pressure.

Non-Return Valves shall have bodies made from bronze for DN not greater than 40 mm and they shall be of the swing pattern type and shall be rated for at least PN 10 or greater as specified. The ends shall be either screwed to EN 10228-1 or flanged to EN 1092-1, PN 10 or as the installation demands.

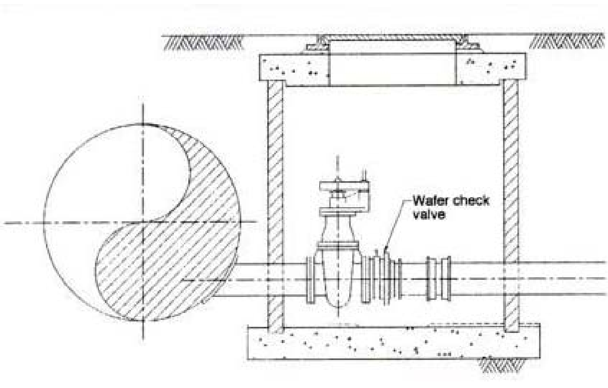
For DN greater than 40 mm they shall be either of a profiled poppet type with non-slam characteristics (surge suppressing type) or of the venture type and shall be of approved manufacture. The proposed valve shall be of low pressure loss and maintenance free with PN 16 rating and shall achieve a movement from fully open to fully closed on pump stoppage in 0.1 to 0.3 seconds. The valve housing shall as a minimum be of epoxy coated cast iron and flanged with the closing system of stainless steel.

With his tender, a Tenderer shall have supplied full technical details of the valves he proposes to supply and install. If the Project Manager deems the valve proposed to be appropriate he will accept the offer. If however the Project Manager considers the offer to be inappropriate he will reject the offer from the Tenderer and instruct him instead as to the acceptable manufacturer(s) of these items. Should a financial offer from a Tenderer in this regard be obviously under-priced then the cost of supplying an appropriate valve shall be fully to the account of the Contractor.

Washout valves should be provided at all dead ends in distribution systems as well as on branch tees at all depressions or low points in pipelines. Their main purpose is to enable the draining down of a section of the main pipeline for repair and cleansing. Primary washouts should be installed to drain to a watercourse the majority length between section valves whilst secondary washouts of smaller diameter can then be fitted to empty any un-drained subsections as required. The tees should be invert tees to ensure that the water and as much sediment as possible is scoured out from the bottom of the pipe whilst primary washout diameters are usually as indicated in the following table and arranged as shown in the accompanying Figure:

TABLE 57: TYPICAL MINIMUM WASHOUT SIZES TO BE ADOPTED

Main Pipeline Diameter	Washout Branch Diameter
Up to 300 mm	80 mm
400 - 600 mm	100 mm
700 - 1000 mm	150 mm
1100 - 1400 mm	200 mm
1600 mm & greater	250 mm



TYPICAL WASHOUT ARRANGEMENT

Washouts should be fitted with soft resilient seated gate valves which must be periodically operated so as to remove recently deposited silt or sediment before it hardens, allowing the water to run out until clear. It is most important that this be done on a regular basis and with sufficient frequency to ensure that any silt and sediment build up does not compact and become difficult to remove by such flushing and to ensure that such deposits do not go septic. Regrettably this is often not done and should a significant build up occur then it may become necessary to dismantle the main pipeline to remove the debris, a time consuming and costly procedure. The discharge from the outlet of the washout must be led to a ditch or stream and the outlet arrangement lined or otherwise protected so that the discharge does not cause scour to the channel leading to the ditch or stream.

4.17.3 Air and Air Valves

The negative effects of air in water pipelines is often grossly underestimated and has led to major problems in many water supply systems. Pockets of air that accumulate at high points and bends create very real pipeline restrictions that lead to considerable loss of pipeline efficiency.

Water normally contains 3%, by volume, dissolved air that can come out of solution in a number of ways.

- Water that is constantly being subjected to changing temperature, flow velocity and pressure, will surrender a surprising amount of air.
- Air is released during the turbulent passage of water through the coarse, tuberculated linings of older cast iron water mains.
- Eddy effect turbulence at bends, valves and other pipeline fittings will release more air.
- Pockets of air will form in the pipeline as a result of the vortex action of pumps.

As well as being released from solution, air can be physically introduced in to water piping. Air can be draw in through pipeline leaks, at damaged joint seals, through leaking valve packing, and through any loose and leaking flange connection.

If there is any breach in the integrity of the pipeline and its fittings, air will enter there. This usually happens when the system is subjected to poorly controlled or unplanned negative pressure events.

The amount of air that can enter the system when the pipes are subjected to negative pressure is often underestimated. Under vacuum conditions, air will find easy entry at any manner of pipeline, or fitting leaks. Vacuum conditions occur far more frequently than many system operators anticipate, and the subsequent damage done at pipe joint seals by negative pressure will permit a significant volume of air to be draw inward.

Air behaves very unpredictably in a pipeline, but even more so in a grid network of pipes. In the normal operation of a water pipeline system, maintenance activities and fluctuating periods of consumption demand will cause air to release from solution and accumulate in the localized piping. Very often, air pockets will form in sections of pipe that are not equipped with air valves, and will travel about the water grid, finding release only when drawn into a water service as someone opens a household tap.

Hence, **Air valves** are a critically important item in all pipeline systems, and as discussed in Chapter 4.11, selection and location of appropriate air valves is most important. For more detailed information, designers are recommended to consult and download the technical papers and utility programme from the website <http://www.ventomat.com/default.asp>.

A suggested air-valve specification is as follows:

Air Release and Vacuum Break Valves

> *"Automatic air relief and vacuum break valves (air valves) shall be of the triple function anti-shock anti-surge type and shall be from an approved manufacturer.*
>
> *Bids which apparently contain non-compliant offers will be required to confirm that they will meet these specifications in their totality at the rates quoted in their offer.*

Such valves will be designed to meet and provide all of the functions described below:

i) Pipeline filling

Uninterrupted high volume air discharge through the large orifice.

ii) Pipeline draining or Column Separation

Uninterrupted high volume air intake through the large orifice.

iii) Pipeline full and operating

Discharge of disentrained pressurised air through the small orifice.

iv) Rapid Filling / Column Separation

The valve must incorporate an integral surge alleviation mechanism that will automatically dampen surge pressures due to rapid air discharge or the subsequent rejoining of separated water columns.

The air release and vacuum break valve shall be of a compact single chamber design with solid cylindrical High Density Polyethylene control floats. These shall be housed in a tubular stainless steel or corrosion protected body with epoxy powder coated cast iron, or stainless steel ends secured by means of stainless steel tie rods.

The valve shall have an integral surge alleviation mechanism which shall operate automatically to limit transient pressure rise or shock induced by closure due to high velocity air discharge or the subsequent rejoining of separated water columns. The limitation of pressure rise must be achieved by deceleration of approaching water prior to valve closure. Relief mechanisms that act subsequent to valve closure cannot react in the low millisecond time span required and are therefore unacceptable.

Large orifice sealing shall be effect by the flat face of the control float seating against a nitrile rubber 'Ó' Ring housed in a dovetail groove circumferentially surrounding the large orifice. Discharge of pressurised air shall be controlled by the seating and unseating of a small orifice on a natural rubber seal affixed to the control float.

The intake/discharge area shall be equal to the nominal size of the valve, i.e. a 150 mm valve shall have a 150 mm intake/discharge orifice.

The valve construction shall be proportioned with regard to material strength characteristics, so that deformation, leaking or damage of any kind does not occur by submission to twice the designed working pressure.

The valve design shall incorporate an over pressure safety feature that will fail without an explosive effect, such as is normally the case when highly compressed air is released suddenly. This feature shall consist of easily replaceable components such as gaskets, seals or the like.

The air valve shall be provided with a separate isolating gate valve or if so specified with a separate isolating butterfly valve.

Unless otherwise specified all air valves shall be provided with an integral flanged inlet with studs appropriate to EN 1092-1 NP 10 or as the installation demands and complying with the appropriate nominal pressure.

All air valves at new installations shall be fitted to an air accumulator tee with the branch of a diameter not less than 0,6 times the diameter of the main. Where necessary, a concentric taper either integral with or as a separate fitting shall be inserted between the branch and the isolating valve immediately beneath the air valve and an abrupt diameter change between branch and isolating valve shall be avoided.“

4.18 PIPE INSTALLATION

It is beyond the scope of this Manual to cover all aspects of pipe installation, this normally being dealt with in detail in Contract Specifications. However, a number of the more crucial points are presented here.

4.18.1 Trench Excavation

Care should be taken when excavating pipe trenches which must be well prepared with provision for minimum cover.

(i) Whilst basic excavation can be undertaken with limited supervision and where appropriate using self-help labour, final excavation and trimming must be done under qualified supervision.

(ii) Trench bottoms must be even without rocks, stones or roots.

(iii) Final excavation and trimming of the trench bottom immediately prior to laying must be undertaken carefully with the use of boning rods to ensure that design gradients are adhered to.

(iv) Minimum width requirements must be maintained to allow for thorough compaction of the upper bedding and for a man to stand with one foot on each side of the pipe for diameters up to DN150 and alongside for larger diameters. This normally means a minimum width of 600 mm and a clearance on each side of the pipe of 300 mm for pipes of a larger diameter.

(v) Trench sides should be vertical and where trench depths are more than 1.5m require temporary shoring during final excavation, bedding and laying of the pipes if there is any risk of trench wall collapse.

(vi) Trench depths must be such that minimum cover requirements of the Design are maintained.

4.18.2 Thrusts on Pipes

All pipelines having unanchored flexible joints require anchorage at blank ends, junctions and bends to resist the thrusts developed by internal water pressure. Permanent thrust blocks and thrust walls must only be cast against undisturbed ground.

Where continuous HDPE pipes are installed, anchor blocks are required when connecting to pipe works of differing materials or fitting with flexible joints and to support valves and hydrants.

The design of anchor blocks shall be based on the actual soil bearing capacity if known, otherwise on an assumed capacity of 75 kPa whichever is less. The inner face of the block shall not be of a lesser thickness than the outside diameter of the fittings, and shall be so constructed as not to impair access to the bolts on the fittings. Concrete shall have a minimum compressive strength of 17.5 MPa at 28 days, and shall be cast against undisturbed surfaces.

A protective membrane to prevent abrasive damage to the water main or fitting shall be provided between the pipe (irrespective of the pipe material) and the concrete thrust blocks.

Thrusts act at right-angles to the flow direction and except at very high velocities, the magnitude of the thrusts may be calculated as follows:

Bends: $F = 10^2 \times A \times P \times \sin ß$ kN

Blank ends and junctions: $F = 10^2 \times A \times P$ kN

Where,

F = thrust force in kN

P = internal pressure in bar

ß = angle of deviation of bend

A = cross –section (m^2) of outside diameter of joint

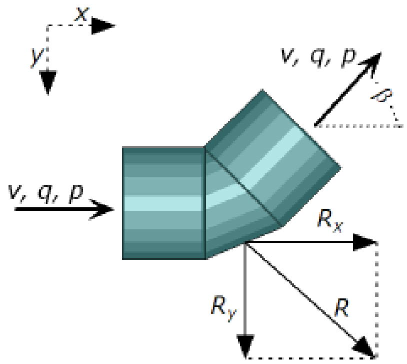


ILLUSTRATION OF THRUST ON BENDS

The following table gives calculated values of thrust forces for standard fittings.

TABLE 4.58 VALUES OF THRUSTS IN STANDARD FITTINGS

NOMINAL SIZE	THRUST PER 1 BAR INTERNAL PRESSURE (=0.33Kg/Cm				
	BLANK ENDS / JUNCTIONS	90^0 BENDS	45^0 BENDS	$22^1/_2{}^0$ BEND	$11^1/_4{}^0$ BENDS
mm	kgf	kgf	kgf	Kgf	kgf
80	76.96	109.07	58.61	30.07	15.29
100	111.11	158.00	85.11	43.83	21.91
150	231.39	327.21	177.37	90.21	4.5.36
200	394.49	557.59	301.73	153.92	77.47
250	601.42	850.15	459.73	234.45	118.24
300	851:17	1202.85	651.37	332.31	167.17
350	1151.88	1625.89	880.73	448.52	225.28
400	1472.98	2084.60	1122.44	574.92	288.48
450	1845.05	2609.05	1411.82	719.67	361.87
500	2268.09	3204.91	1732.92	883.79	444.44
600	3226.29	4566.76	2471.96	1259.93	633.02
700	4362.89	6167.17	3338.43	1701.32	855.24
800	5677.88	8027.52	4342.50	2215.08	1113.14
900	7150.86	10112.13	5474.00	2790.01	1401.63
1000	8792.04	12436.28	6727.82	3441.19	1723.75
1100	10626.91	15025.48	8134.55	4145.76	2083.58
1200	12609.58	1728.74	96::3.41	4920.48	2471.96
1400	17110.09	24199.79	13098.37	6676.86	3353.72
1600	22273.19	31498.47	17043.83	8691.13	4366.97

4.18.3 Pipe Handling and Storage

During handling, all pipeline components shall be protected from damage and shall be transported and stored in such a way that they do not come into contact with hazardous substances. Equipment suitable for loading and unloading shall be used and all work must be carried out in accordance with National Health and Safety Regulations.

On delivery to site, all pipes should be checked for damage prior to acceptance and all accepted pipes and components shall be stored and handled to prevent damage.

Storage areas shall be flat ground free from rocks and other damaging materials, well drained, and free from chemicals which might be detrimental to coatings, linings, or pipe wall materials. All PVCu pipes shall be stored under shading to prevent UV deterioration and at temperatures not exceeding 25^{o}C.

Pipe ends should be sealed to prevent contamination and pipe sockets should not be in contact with the ground but packed up from it. Pipes should be adequately supported along their length and stack heights must not exceed that recommended for each material.

Whilst ferrous pipes may be strung out along trench sides in advance provided they are properly supported off the ground, PVC pipes are sensitive to UV light, head and temperature and care must be taken when bringing them to the trench side and they should be strung out along the side of pipe trenches not more than one full day before laying and backfilling at which time restrictions on shading and temperature can be relaxed.

At the trench side, pipes should be supported so that they are off the ground. Re-usable hessian bags filled with sand or sawdust are suitable for this purpose.

4.18.4 Pipe Laying

4.18.4.1 Pipe Laying Specifications

It is beyond the scope of this Design Manual to provide detailed specifications for Pipe Laying, besides which most Designers and Consulting Engineers will have access to their own Specifications developed over time and modified based on experience.

However and as a guideline, the following points should be addressed in such specifications.

4.18.4.2 General

Care should be taken when laying pipes and trench bottoms must have been finally prepared and trimmed only shortly before laying. The laying, jointing and proper embedment of pipelines is crucial for the attainment of working life and probably the most neglected and abused water supply construction activity.

(i) Pipes should be laid along the straightest possible route and crossing of roads, railways creeks and rivers should be done at right angles wherever possible.

(ii) If Horizontal Directional Drilling (HDD) beneath roads and railways is undertaken with pipes laid subsequently in sleeving, then the sleeved pipe section must have been designed accordingly (Support factor = 1) as there will be no compacted embedment material around the pipe in the sleeve.

(iii) The lower bedding layer must be placed and compacted first to meet the specified minimum density and be of the required material and grading as specified in the Design.

(iv) Before laying the pipe onto the compacted lower bedding, space must be prepared at joints to allow the pipe to be fully supported on the lower bedding along the full length.

(v) The upper bedding must be placed and compacted evenly on either side of the pipe again in full compliance with the Design. Care must be taken in placing the upper bedding that it is fully beneath the pipe and there are no voids. Care must be taken during compaction to avoid damage to the pipe or its coating.

(vi) All subsequent backfilling should be completed in layers and there must be complete and thorough compaction of the embedment to achieve as a minimum, the level of compaction called for in the Design.

(vii) Air valves should be provided at high points to allow for escape of air. The tee branch feeding the air valve should be dimensioned as an air-accumulator branch.

(viii) Washouts should be provided at low points to allow for periodic flushing of the mains.

(ix) Other than long radius horizontal directional changes taken up at flexible joints, changes of direction should be done using prefabricated bends or for PE pipes only by bending or curving the pipe.

(x) Before backfilling above the embedment layer of thermoplastic pipes, a metallic locational tape should be laid to make future location possible.

4.18.4.3 Pipe Testing, Flushing and Disinfecting

Pressure testing of the pipeline before the final backfilling of the trench is most important. However the embedment layer should be completed such that the pipe is covered in fully compacted material with a minimum thickness over the crown of the pipe of 300 mm. Whilst PVC joints should be covered or otherwise protected from UV light, Steel and D.I. joints can be left exposed until each pipe section is satisfactorily tested provided that expansion and contraction variations in length can be absorbed.

All anchor and trust blocks should be in place and fully cured and the pipeline fully secured before testing. Where pipelines cannot be tested in one operation they shall be tested in sections, whilst the joints linking the individual sections shall be tested for leaks by a final overall test.

Test pressures should be 20% to 50% higher than the service (working) pressure. Some pipes are manufactured with a test pressure equal to twice the working pressure but at the lowest point of the pipeline it should never be higher than 1.2 times the nominal pressure of the pipes.

A hand pump fitted with calibrated pressure gauge should be used graduated to permit the correct reading to 0.1 kg/cm2 pressure changes. The pump arid gauge should be placed at the lower end of the section. The testing pressure shall last for 15 minutes for every 100m of pipeline. The air at high points should be released during the filling of the pipeline for testing.

Once a pipeline has passed its pressure test and before any pipeline is put into service it must be thoroughly flushed out through the washouts and disinfected.

Then and only then is the pipeline ready to be put into service.

4.19 LIFE CYCLE ASSESSMENT AND WHOLE LIFE DESIGN

As introduced in section 4.2.5, ‘Life Cycle Assessment’, ‘Cradle to Grave’, or ‘Whole Life Design’ are terms that describe the costing of sustainability in the design, manufacture, installation, operation & maintenance, repair, and eventual decommissioning and where appropriate re-use of a built asset or its properties. It entails achieving compromise and synergy between economics, environmental and social costs.

In this particular case it requires the consideration of all costs and values of a pipeline in its design and selection. It requires the consideration and as far as practicable, the cost quantification of numerous factors, as listed in section 4.2.5, some of which are frequently changing due to external factors and in particular due to such things as international demand for the raw materials involved and oil prices.

As such it is an extremely complicated process with many interactions and difficulties in the compilation of basic data.

In breaking the life cycle down into processes, it is not always clear how far one should go in including processes belonging to the product concerned.

In the production of polyethylene, for example, oil has to be extracted; this oil has to be transported in a tanker; steel is needed to construct the tanker, and the raw materials needed to produce this is steel which also has to be extracted. For practical reasons a line must be drawn.

PVC production also needs oil but in addition requires the electrolysis of salt to produce chlorine. The environmental effects of the electrolysis process cannot be ascribed entirely to chlorine alone, as caustic soda and hydrogen are also produced. A suitable allocation rule is needed here, for instance allocation on mass basis or economic value of the products.

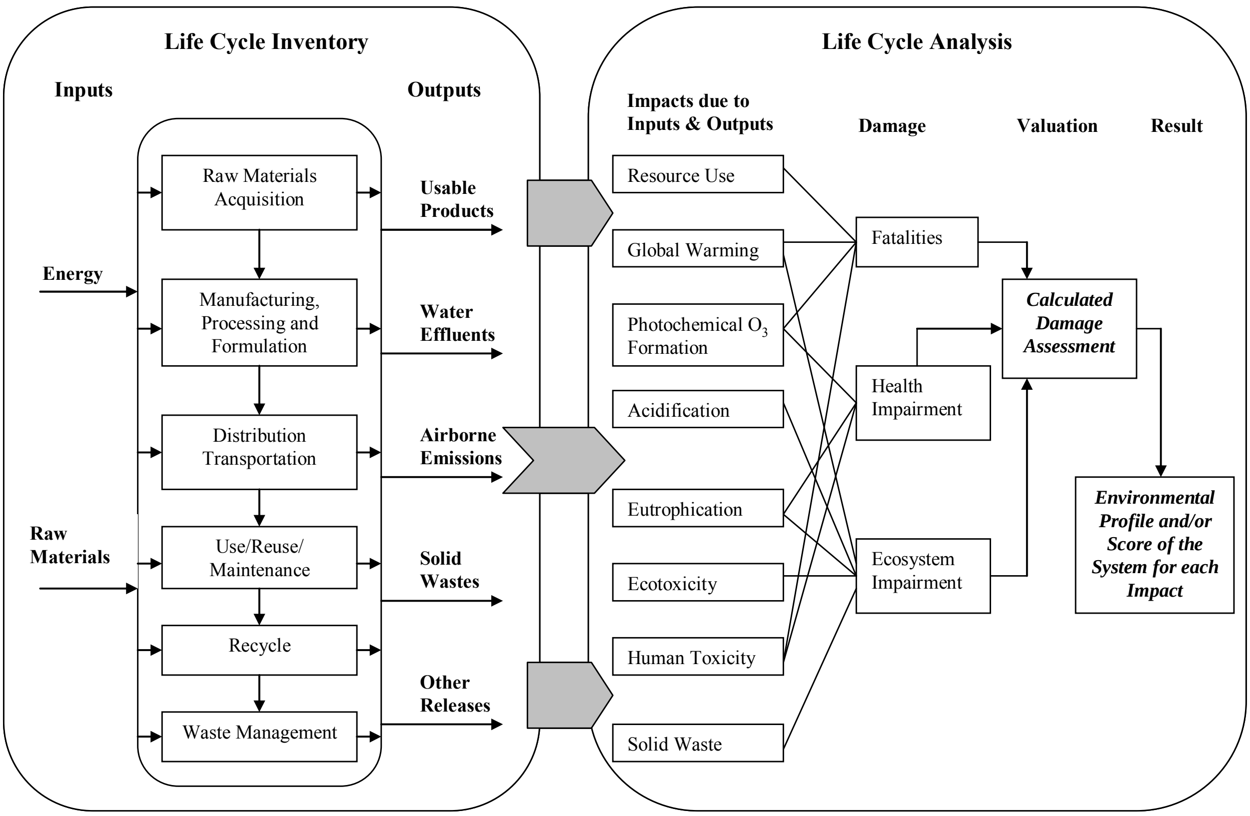
An overview of what is entailed is shown on the figure on the following page.

As can be seen, this is sufficiently daunting to deter many from attempting such an analysis.

Guidelines for LCA are provided by the ISO 1404x series who in 1996 released new, improved editions of its life cycle assessment standards designed to highlight environmental problems and areas for improvement in the production and use of products.

These new standards facilitate the process of evaluating the impacts that a product has on the environment over its entire life, thereby encouraging the efficient use of resources and decreasing liabilities.

Life Cycle Assessment: An Overview



The goal of LCA is to compare the environmental performance of products in order to be able to choose the least burdensome. The term 'life cycle' refers to the notion that for a fair, holistic assessment the raw material production, manufacture, distribution, use and disposal (including all intervening transportation steps) need to be assessed. This then is the 'life cycle' of the product. The key ISO standards are:-

ISO 14040:2006, Environmental management – Life cycle assessment – Principles and framework. This provides a clear overview of the practice, applications and limitations of LCA to a broad range of potential users and stakeholders, including those with a limited knowledge of life cycle assessment.

ISO 14044:2006, Environmental management – Life cycle assessment – Requirements and guidelines. This is designed for the preparation of, conduct of, and critical review of, life cycle inventory analysis. It also provides guidance on the impact assessment phase of LCA and on the interpretation of LCA results, as well as the nature and quality of the data collected.

In the case of deciding on suitable pipewall materials for Tanzania the designer is clearly dependant on the information provided by the international pipe industry or the analyses undertaken by or on behalf of others.

The international pipe industry is supported by a number of country associations representing the different types of pipe from which a professional can choose. However, most of these associations' members are manufacturers, joined together in the common goal of promoting their type of pipe over others. Each group understandably has a vested interested in its own type of pipe, each offers a wealth of resources for pipe specifiers to consider when trying to make a balanced, well-informed decision. However, caution must be used in separating fact from rhetoric.

Value judgements are often presented as facts and unbiased hard quantifiable data is still difficult to obtain. At present, vested interests and difficulties in putting costs on such things as environmental hazard make it impossible for decisions to be made on numerical data alone.

In the international water industry, where a limited number of comparative studies have been made there is more attention to sewer pipes than to water pressure pipes and even when independent, they remain inconclusive [< >].

One result has been to first consider energy inputs during manufacture and installation rather than a complete cycle to grave approach. Problems during operation and maintenance and other long term environmental consequences of manufacture and more importantly of disposal are rarely discussed by the particular industry concerned.

Also there is as yet very limited published information from water supply utilities worldwide on the real operational and maintenance costs of pipelines based on different pipe wall materials, even in Europe and North America and very few of such organisations as yet use Activity Based Costing techniques which is considered necessary for a comprehensive LCA.

A simplified approach is therefore necessary at present, and the following is considered appropriate for Tanzania at the present time.

[< >] Life Cycle Assessment of PVC and of Principal Competing Materials, Final Report commissioned by the European Commission, July 2004
This did not consider pressure pipes, was largely qualitative, said to be due to lack of acceptable comparable data and the pipe comparison was limited to PVC, PE, concrete and fibre cement.

TABLE 4.59: RECOMMENDED SIMPLIFIED LCA APPROACH FOR TANZANIA.

Item	Type	Approach to be Adopted
Supply	Economic; direct	A full and comprehensive evaluation based on competitive bidding, or best assessment (Force Account Projects) based on rigorous but fair specifications with minimum pipe cover taking into account likelihood of vandalism and inclusion of necessary embedment materials and compaction
Install	Economic, direct	
O & M	Economic, indirect	Use Client's data if available, otherwise consider key parameter costs such as burst risk from elsewhere (see below)
Environmental	LCA	Detailed consideration of information on embodied energy and long term impact risks on human health (see below)
Social	LCA	Use Client's data if available, and make a general statement on possible consequences of using different pipes
Refer to section 4.2.5, pages 4-5 and 4-6 for details of what needs inclusion		

In effect this approach then becomes as close as practicable a total cost of ownership approach to which is then added a level of consideration for the environmental and social consequences based on readily available current information.

a) **Economic Direct Costs.** These have been dealt with extensively elsewhere in this chapter.

b) **Economic Indirect Costs.** Use any reliable comparative data that the Client has or that can be obtained from similar sized utilities elsewhere in East Africa, or if lacking based on the following or similar information.

Usually, the most expensive maintenance item is that of attending to pipe failure and here there is some limited guidance. Figures from the UKWIR National Failures Database for the years 1998 to 2001 are given in the following table. It must however be borne in mind that the average age of the pipes in the UK comparison here varies with steel being the oldest, followed by DI and PVC and more recently PE and that particularly with steel pipes, corrosion when coatings were far less developed than today, was a significant cause.

TABLE 4.60: AVERAGE ANNUAL FAILURE RATE PER 100 KM

Location	PE	DI	Steel	PVC
UK, actual 1998-2001	3.2	4.9	5.6	8.3
Recommended, for Tanzania	3.0	5.0	5.5	10.0

These figures should be converted into a repair cost over the design life of the pipeline and capitalized for comparison purposes.

c) **Environmental Costs and Implications.** The quantitative information provided here is probably the best currently available at the time of revising this chapter (end 2006). However it is under a state of near continuous improvement and refinement and where possible designers are encouraged to obtain updated information and use that instead.

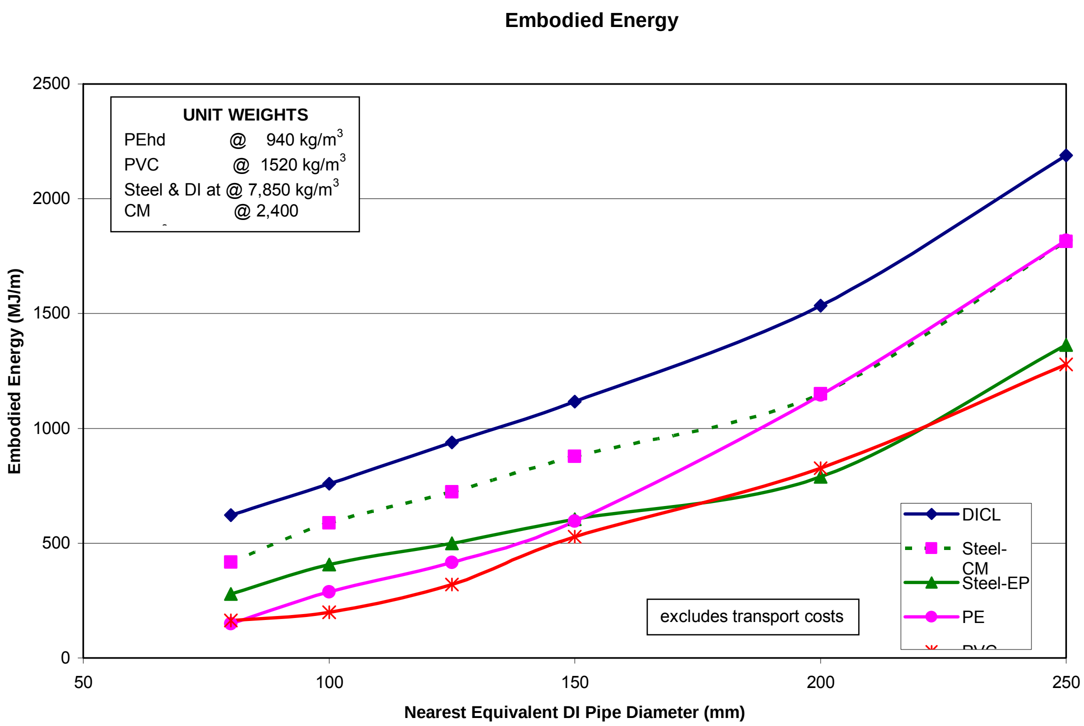
Because of climate change, it is clear that **energy efficiency** is an important factor and here the concept of Embodied Energy Analysis (EEA) has developed as a first step towards a full environmental LCA component. Embodied energy is the energy consumed by all of the processes associated with a product, from the acquisition of natural resources through manufacture to product delivery. This includes the mining and manufacturing of materials and equipment, the transport of the materials and associated administrative functions. It is usually given in MJ/kg. Unfortunately figures quoted by different organisations vary considerably and are often country specific. The figures given in Table 4.60 are however useful as an initial guide. For purposes of comparison, such figures need to be converted into running metres for each comparable diameter of pipe as illustrated in Table 4.61.

TABLE 4.61: EMBODIED ENERGY COEFFICIENTS FOR DIFFERENT PIPE TYPES

Pipe Type	Embodied Energy (MJ/kg) CIRIA (Australia) 2002	Embodied Energy (MJ/kg) Buchanan 1994
Ductile Iron Pipe	38.2	
DICL (cement/mortar lined)	40.2	
Steel Pipe		56.9 *
PVC-U	74.9	
PE100	75.2	

** Based on a virgin steel value of 30.0. Add 2.0 if cement mortar lined. The effects of 3LPE and polyethylene coatings for Steel & DI respectively are minor in comparison and less than the inherent approximations in the above. For transport (shipping costs), add 0.25 MJ/tonne-km.*

The results of compiling this energy information are presented in the graph below and in Table 4.62 on the following page. This indicates that up to and including DN150 mm, there is no major difference between PVC, PE and epoxy lined Steel [< >], with PVC always the lowest energy demanding material whilst CM lined steel is distinctly higher and D.I. (DICL) higher still. At DN 200 mm and DN 250 mm, PVC and epoxy lined Steel have very similar demands (excluding transport) as do CM lined steel and PE although both are some 40% higher, with DICL remaining quite significantly the most energy demanding.



GRAPHICAL COMPARISON OF EMBODIED ENERGY COSTS

< > For steel diameters less than DN 600, epoxy spray rather than FBE has to be used and this brings with it some environmental consequences during manufacture which need to be taken into consideration.

TABLE 4.62: COMPARATIVE WEIGHT/METRE & EMBODIED ENERGY

DN (DI)	Pipe	Nearest (mm)	Pressure (PFA)	Wall thick-ness (mm)	Lining (where applicable) (mm)	Internal diameter (mm)	Weight (Kg/m)	Embodied energy (MJ/m)
80	DICL	98	85	6.0	3.0	80	15.49	623
	Steel	88.9	60.8	2.3	4.0	76.3	7.33	417
	Steel	88.9	60.8	2.3	0.0	84.3	4.91	279
	PE	90	10	8.2	0.0	73.6	1.98	149
	PVCu	90	12.5	5.4	0.0	79.2	2.18	163
100	DICL	118.0	85.0	6.0	3.0	100	18.90	760
	Steel	114.3	53.5	2.6	4.0	101.1	10.33	588
	Steel	114.3	53.5	2.6	0.0	109.1	7.16	408
	PE	125.0	10.0	11.4	0.0	102.2	3.82	288
	PVCu	110.0	12.5	5.3	0.0	99.4	2.65	198
125	DICL	144.0	85.0	6.0	3.0	126	23.34	938
	Steel	139.7	43.7	2.6	4.0	126.5	12.73	724
	Steel	139.7	43.7	2.6	0.0	134.5	8.79	500
	PE	160.0	14.6	12.7	0.0	134.6	5.52	415
	PVCu	140.0	12.5	6.7	0.0	126.6	4.26	319
150	DICL	170.0	79.0	6.0	3.0	152	27.77	1116
	Steel	168.3	36.3	2.6	4.0	155.1	15.42	878
	Steel	168.3	36.3	2.6	0.0	163.1	10.62	605
	PE	180.0	10.0	16.4	0.0	147.2	7.92	596
	PVCu	180.0	12.5	8.6	0.0	162.8	7.04	527
200	DICL	222.0	62.0	6.3	3.0	203.4	38.18	1535
	Steel	219.1	27.9	2.6	4.0	205.9	20.21	1150
	Steel	219.1	27.9	2.6	0.0	213.9	13.88	790
	PE	250.0	10.0	22.7	0.0	204.6	15.24	1146
	PVCu	225.0	12.5	10.8	0.0	203.4	11.05	827
250	DICL	274.0	54.0	6.8	5.0	250.4	54.44	2188
	Steel	273.6	31.0	3.6	4.0	258.4	31.88	1814
	Steel	273.6	31.0	3.6	0.0	266.4	23.97	1364
	PE	315.0	10.0	28.6	0.0	257.8	24.19	1819
	PVCu	280.0	12.5	13.4	0.0	253.2	17.06	1278

Steel wall thickness from ISO 559. Whilst CM equates to about 60% of the weight of a finished class B steel pipe, epoxy is of the order of 2% by weight and polyethylene of the order of 7%.
Unit weights used: PVC 1520 kg/m^3, PEhd 940 kg/m^3, DI & steel 7850 kg/m^3, CM 2400 kg/m^3.

When it comes to the **environmental health implications** of the four pipe materials considered here, unquestionably the most noxious is PVCu and its additives, followed by PE, whilst both DI and steel cause the least environmental problems.

In comparison with PVCu, PE uses fewer problematic additives, has reduced leaching potential in landfills, and reduced potential for dioxin[< >] formation during burning (provided that brominated/chlorinated flame retardants are not used), and reduced technical problems and costs during recycling.

[< >] Dioxins are considered a global health threat because they persist in the environment and can travel long distances. At very low levels, near those to which the general population is exposed, dioxins have been linked to immune system suppression, reproductive disorders, a variety of cancers, and endometriosis.

Some problems have been reported for cement mortar entering into solution and for sprayed epoxy but these again are minor compared to the inherent environmental risks associated with PVC during the manufacture of the resin and in its eventual disposal.

Problems with sprayed epoxy reportedly relate to minor skin problems (eczema) and as with any paint type of spray if inhaled, with risk of respiratory problems. Protective masks should therefore always be worn in such environments.

Manufacture of PVC resin is now very tightly controlled and dioxin release is significantly lower than it was in the early years of production. Lead and organotins as additives are banned in Australia, not used in potable water pipes in the USA and being progressively phased out in Europe.

However disposal of PVC is what must be of main concern in countries such as Tanzania, even though dioxins are spread worldwide in the atmosphere.

There are two schools of thought on the most effective means of disposal of PVC waste but in all cases it is recognised that burning at low temperature whether deliberate or accidental should be avoided at all costs. That is the dilemma facing Tanzania.

In the USA, disposal to carefully controlled landfill sites is regarded as acceptable because land is plentiful and the risk of spontaneous combustion due to other landfill elements is rare under the control disposal conditions imposed. In Europe where land is scarce, high temperature combustion at or above 900^{o}C is the preferred method as the dioxins are destroyed at such temperatures.

For Tanzania, present and future disposal is likely to create major problems for the health of future generations and the designer must not only be aware of this but ensure that the Client is also.

Under such circumstances this Manual cannot give specific guidance but can only suggest that where practicable, the choice should be between locally produced PEHD and locally produced barrier coated Steel.

d) **Social Costs and Implications**

When it is agreed between Financier and the Government to provide a local preference in order to promote employment and industrialisation, this is a social cost to be included.

Other social costs may be unquantifiable for now but if the Client is able to provide usable information this should be included.

4.20 WATER STORAGE

Storage tanks are needed to balance the variations in the water consumption during the day; to ensure a continuous supply during power supply outages and breakdowns of pumping plant and lastly to give capacity for exceptional demand e.g. fire fighting. The storage volume should be large enough to accommodate the cumulative differences between water supply and demand.

4.20.1 Service Reservoirs

The balancing reserve is defined as a quantity of water required to be stored in order to serve out the variation in demand in the distribution system against the constant supply from the intake or treatment works. Balancing storage provides water when demand exceeds the maximum hourly demand on which the feeder pipe is based. The amount of this storage should be such that it can meet the difference between peak day demand and peak hour demand for certain times.

For proper dimensioning of the balancing storage the pattern of the daily per hour consumption (daily hydrograph) must be known. If no records are available, experiences from systems of the same size and kind can be used as a guide.

The consumption for rural areas can be assumed to occur in the morning between 6am and 9am (06.00 and 09.00 hours) and in the afternoon between 3pm and 6pm (15.00 and 18.00 hours). Since the consumption during these hours is assumed to be constant, the dimensioning~ flow is about 17% of the average daily demand. With a constant flow from the intake the balancing storage should theoretically be 37.5% of the daily demand.

Knowing or assuming the mass - curve of the consumption, the capacity of the balancing storage is then found by combining the mass - curve of consumption and mass - curve of production. The maximum deficit added to the maximum surplus then gives total capacity needed.

For a pumped water supply, a different number of pumping hours will give different capacities of balancing storage required. It is up to the Designer to optimise the capacity of the pump and rising main with the capacity of the balancing reservoir. Where the distribution system is designed for direct pumping into the system, it is an advantage to provide a balancing tank at the end of the system with a nominal capacity (1-2 hours) to provide pressure relief and improve the overall flow and is called a reservoir floating on the line.

4.20.2 Emergency Reserve

In order that normal maintenance work may be carried out on the mains, intakes or treatment plant, a certain extra storage should be provided to allow for an uninterrupted supply during this maintenance period. Storage should also be provided to cover breakdowns of pumping plant and power supply cuts. The magnitude of this storage component depends on the likely nature of the interruptions and the time required to bring the supply back to normal.

During this period there should be enough storage to meet the total hourly fluctuations of the demand as there is no inflow taking place. In order to minimize the need of emergency reserve, standby units for energy and pumping should be considered. On pumping mains, normal maintenance can be carried out during the periods when the pump has stopped. Treatment units also should be two in line to facilitate maintenance without interruption of the supply.

With the availability of standby power and pumping units, parallel lines of treatment plant etc. the emergency storage becomes a function of the time required to carry out repairs to the feeder main only. This time could range from some minutes to a few days depending on the nature of the breakdown, the location and the preparedness, efficiency and experience of the maintenance team.

For practical design purposes, emergency reserve should be taken between 3 to 8 hours of the peak day demand.

4.20.3 Fire-Fighting Reserve

The quantity of water required to be stored for fire fighting purposes is the product of the designed fire flow and the probable duration of the fire. These factors have already been discussed in the section 4.6.5.

The figures given apply only for areas of normal residential, commercial and public use. Industrial and other areas with high fire risk potential like airports, oil depots, big hospitals etc. must be considered on a case by case basis. The principle for their fire fighting arrangements should be that the fire fighting reserve for those areas be provided separately.

4.20.4 Design Aspects

The total capacity of water storage is the sum of balancing, emergency and fire fighting reserve. Normally this total lies between 30% and 50% of the peak daily demand and for rural water supplies a figures of 50% is normally adopted. Preferably the depth of storage tanks of normal size say up to 1000 m^3 should not be less than 3 metres or higher than 6 metres.

To allow for a continuous supply during maintenance and repairs to storage tanks consideration should be given to either providing two reservoirs parallel in line or installing a by-pass pipe. This latter solution cannot be used when the storage tanks also acts as a break pressure tank.

Inlet and outlet piping for the tank should be so located that a good mixing of the water occurs. Usually the inlet pipe is brought to the same level as the over flow pipe. The overflow pipe should be fitted with a strainer and is normally connected to the washout pipe beyond the washout pipe control valve.

One or two access manholes with locking arrangements should be provided according to the size of the tank. Coated steel or aluminium ladders below the manhole are provided to climb down for tank cleaning purposes. Adequate ventilation should be provided in the roof slab with long-lasting safe guards against animal and insect intrusion. Where reservoirs also provide retention time for chlorine disinfection there will be a heightened risk of corrosion and iron and steel ladders and air vents should be avoided. All internal and external steel work, non barrier-coated metal piping and specials should be painted with several coats of red lead and bituminous or aluminium paints.

A water level indicator should also be provided. All valves should be positioned in strong and lockable valve chambers with adequate room for repairs and replacements. In that respect it flanged valves should be provided with a flange adapter on the non-thrust side of the valve.

4.20.5 Location

Determining the location of a storage tank is governed by two main factors, namely the topography of the area and the distance to the supply areas.

Since an elevated tank is about 40% more expensive than a ground level tank, the latter is the first choice and a suitable high rocky point or other location should be selected. A careful cost comparison should be made between an elevated tank and a ground level tank requiring extra pipe lengths. Whilst the final decision depends on such a comparison, in general a ground level tank is preferred.

Further important considerations are the minimum pressure at the highest supply point and the maximum pressure at the lowest point.

These are to be 5m and 60m water height respectively. Thus at no point in the main should the residual head should be less than 5m or more than 60m and when otherwise it is lower, a booster arrangement should be introduced. In most cases, when the storage tank is between the source and the service area or situated beyond (past) the service area, it is advantageous to use one main only, acting as a supply (gravity or pumped) main to the tank and as a distribution main to the service area.

The storage tank is simply connected to this main by an equal tee and pipe into the tank. However this connecting pipe branches again into two pipes, one branch entering the tank at top water level and the second branch exiting at floor level and provided with the normal gate valve and a non-return valve to prevent water flowing back into the tank via the outlet.

This arrangement avoids in many cases two parallel pipelines and the need to provide a by-pass to the tank. This of course is dependant upon the fact that no supply point in the service area should be subjected to higher static pressures than 60 m. To achieve a very efficient distribution system where the people living in low level areas and high level areas equally get their required supply, the elevation of the tank should be such that in terms of pressure there should not be much difference in the service pipes supplying low level and high level areas of the town.

If the topography of the supply area varies considerably, then the service areas must be zoned according to the ground level and demarcated as (say) a low level, middle level and high level zone. Each zone should have its own distribution system along with its own smaller reservoir supplied by a separate feeder main from the single main service reservoir.

By having separate reservoirs, the cost of constructing one large reservoir overall is avoided and the zoned reservoir are positioned according to the elevational requirement of the respective zone. The separation of distribution systems in this way functions very satisfactorily if ground level (piezometric pressure) variation is not very much.

The cost of the distribution system may also reduce due to the use of smaller and lower pressure class pipes although as noted elsewhere, for thermoplastic pipes in particular, it is not the hydraulic pressure alone that determines wall thickness and hence pressure class. In such instances, the Designer must make tentative designs to compare the cost of:-

(1) One large reservoir with one distribution system,

(2) One large reservoir with three primary distribution mains and three different distribution systems, and

(3) Three small reservoirs at different elevations and with three individual distribution systems.

Sometimes two zones may be sufficient according to the topography. The designer should choose the best alternative both economic and efficiency wise.

Different distribution zone mains are normally inter-connected with lower zones to facilitate supply of water from one zone to the other during times of emergency and water shortage.