FERROCEMENT RESERVOIRS – A South African Perspective

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WATER RESEARCH COMMISSION

TT 499/11

Ferrocement Reservoirs
a South African perspective

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Report to the

Water Research Commission

by

Partners in Development

WRC Report No. TT 499/11

October 2011

Obtainable from:
Water Research Commission
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The publication of this report emanates from a project entitled *An evaluation of the durability and cost effectiveness of ferrocement reservoirs* (WRC Project No. K5/1818).

Cover photograph: 40 kℓ ferrocement reservoir constructed in 1992, photographed during this research in July 2008. KwaNyuswa area, Ndwedwe, Ilembe District Municipality, KwaZulu-Natal, South Africa.

ISBN 978-1-4312-0142-6
Printed in the Republic of South Africa

EXECUTIVE SUMMARY

Ferrocement is the name given to a composite material made from a high strength mortar cement mix reinforced with steel mesh and wire. It is distinct from reinforced concrete in the following ways:

i) no stone aggregate is used in the mix,
ii) the steel reinforcing consists of small diameter wires and mesh closely spaced, rather than larger diameter bars spaced further apart,
iii) minimum thicknesses of ferrocement elements are small, from as little as 10 mm,
iv) ferrocement is not poured between and moulded by shutters, but is worked by hand into the steel mesh reinforcement,
v) due to this construction method ferrocement can be moulded into any shape.

In its pure form ferrocement has a steel content of over 5%, which would be very high for reinforced concrete. The total steel content is, however, not necessarily higher because ferrocement structures are thin wall structures.

The first recorded use of steel in combination with cement is attributed to the Frenchman Joseph Louis Lambot, who in 1848 made a rowing boat using cement mortar reinforced with steel bars and wire mesh. Before long the new material had been used for items as diverse as flower pots and guard rails. The French called this new material “ferciment”. Soon after this an Englishman named Wilkinson realised the value of steel reinforced concrete beams in buildings, beginning a revolution in building design on which all modern structural concrete engineering is based. With this being the main commercial application and with steel rods being more easily available than steel mesh, the use of steel mesh in reinforcing thin shell structures was for a period forgotten.

Ferrocement, as distinct from reinforced concrete, was rediscovered by the Italian Pier Luigi Nervi in the 1940s. He used the material for moulded thin shell roof construction, and also built ocean-going boats from ferrocement, spawning the “concrete yacht” boat industry. It was not long before the value of ferrocement in the construction of water retaining structures was realised. With its high cement content and low cement-water ratio, the mortar used in the construction of ferrocement is significantly less permeable than ordinary concrete. Furthermore the thin shell nature of the construction means that less material is used than is the case for a reinforced concrete reservoir of similar size. SB Watt in his 1978 book *Ferrocement Water Tanks and their construction* describes ferrocement water tanks built since the 1950s in the United States, New Zealand, South East Asia and Southern Africa (specifically in Botswana and the then Rhodesia (now Zimbabwe).

While ferrocement boats were being built in South Africa from the 1960s, another two decades passed before the introduction of ferrocement for water tanks or reservoirs. Initially its use was restricted to small reservoirs (typically 5 000 litres) used in conjunction with spring protections and rain water harvesting by NGOs engaged with rural development work such as the Valley Trust and World Vision. The first known use of ferrocement for a larger reservoir in South Africa was by the CSIR (Council for Scientific and Industrial Research) who used it for two 40 kℓ reservoirs at KwaHlophe in the Ndwedwe District of KwaZulu-Natal in 1990. In 1992 the CSIR again used ferrocement construction for two 75 kℓ reservoirs at the neighbouring KwaNyuswa Water Project, as well as a number of smaller reservoirs. In 1993 the CSIR built a number of 100 kℓ ferrocement in Maphumulo, the neighbouring district.

The first CSIR ferrocement reservoirs were based on a design which had previously been used by Graham Simpson, then an engineer with the CSIR’s Building Research Division, when he had worked in the then Rhodesia. Simpson’s design incorporated a catenary shaped roof, which the CSIR changed for a fibreglass roof which was not a success. At kwaNyuswa a domed mortar roof design was used and this worked well. Since the early 90s several hundred ferrocement reservoirs in the size range 5 kℓ to 220 kℓ have been built in South Africa based essentially on the design first used by the CSIR at KwaNyuswa. The largest ferrocement reservoirs built to date in South Africa were two 450 kℓ tanks built in 1995 at

Osindisweni, some 20 kilometres west of Verulam. These were designed by the consultants James Crosswell and Associates and built by Exter Construction.

Large ferrocement reservoirs have therefore been in use in South Africa since 1992. Based as they are on a thin shell design they are typically 40% cheaper to build than standard reinforced concrete. What most engineers want to know, however, is whether they can be trusted to stand up in the long term. One of the objectives of this study, therefore, was to carry out field evaluations of ferrocement reservoirs in practice to see how they had weathered in the field. A total of 41 reservoirs were visited and they were checked for signs of spalling, cracking or leakage. It was found that the reservoirs were in general performing well and that they had not undergone any discernible deterioration.

It is normal for ferrocement reservoirs to display a certain amount of microcracking. This is because ferrocement is thin walled and relatively flexible. Mostly these cracks are superficial and limited to the outside of the reservoir, but where they do seep or weep they generally seal through a process of calcification within a period of weeks or months. Calcification and streaking is common with all water retaining structures, up to the largest dams. For those who are not familiar with this material this cracking may be disconcerting. For this reason the application of two coats of a good quality PVA on the outside of the reservoir, refreshed every few years, greatly improves the reservoirs' outward appearance and is recommended.

Over the years the following design changes have been introduced:

- uPVC piping, which does not rust like galvanised steel piping, was used for all pipes cast into the base. [A watertight bond is achieved by scouring the outside pipe wall, or using PVC glue and sand]
- The shape of the roof shuttering (previously shaped by eye and therefore sometimes irregular) was regularized using a support system of telescopic poles and interlocking radial rings. This change straightened the roof slope somewhat so that the new roof design, though still convex, was more cone than dome shaped.
- More wire and mesh was added to certain sizes of reservoir to increase the design safety factor and reduce micro-cracking. In particular the importance of vertically oriented steel in the smaller reservoir sizes has been proven and this is now standard for all but the smallest (5 kℓ) reservoirs, where wire and fowl netting suffice.
- A bandage and sealant was specified as standard on the wall-floor joint.

Design parameters have been developed for the walls and the roofs, and these have been included with this report. A finite element analysis has been carried out to check the accuracy of the simplified method used for the analysis of the roof stresses, and this has proven the adequacy of the design method used.

A construction manual is also included with this report, and the key points have been highlighted. Like any craft ferrocement construction can be done well or badly and attention to detail is important.

Given their significantly lower cost and the evidence of their durability, there is no reason why ferrocement reservoirs cannot be used with confidence. They do not necessarily look as smart or professional as reinforced concrete reservoirs which have the cleaner and straighter off-shutter finish, but particularly in the agricultural and the rural sector, where cost is a greater consideration, they have their place. A further advantage in a country with South Africa's high unemployment levels, is that they are more labour-intensive than other types of reservoirs, and they do not require the importation of any expensive materials.

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1. INTRODUCTION

One of the greatest achievements and conveniences of modern civilization is piped water supply. Wherever such supplies are built, safe hygienic bulk storage of water is required. Various methods are used for the construction of water reservoirs, including:

- Reinforced concrete
- Plastic
- Fibreglass
- Fibre cement
- Plastered brick masonry, reinforced with steel
- Brick masonry in combination with reinforced concrete
- Plastered stone masonry, reinforced with steel
- Earth lined with a rubber membrane and covered with a floating membrane
- Corrosion protected steel with or without plastic or rubber membrane liners
- Timber reinforced with steel
- Ferrocement

Of the above, reinforced concrete is the most durable and is for this reason preferred by engineers, but it is expensive. Figure 1 shows current costs for reinforced concrete reservoirs derived from tender prices on recent construction projects in KwaZulu-Natal, South Africa (Partners in Development, 2010). As can be seen below the unit cost (i.e. Rand per kℓ of water stored) of reinforced concrete reservoirs escalates steeply for the smaller reservoir sizes. It is in the 5 to 200 kℓ size range that alternative and less expensive types of reservoirs are of particular interest. This is also the size that is most commonly used in small village scale water supply systems and for distributed and secondary storage in larger systems.

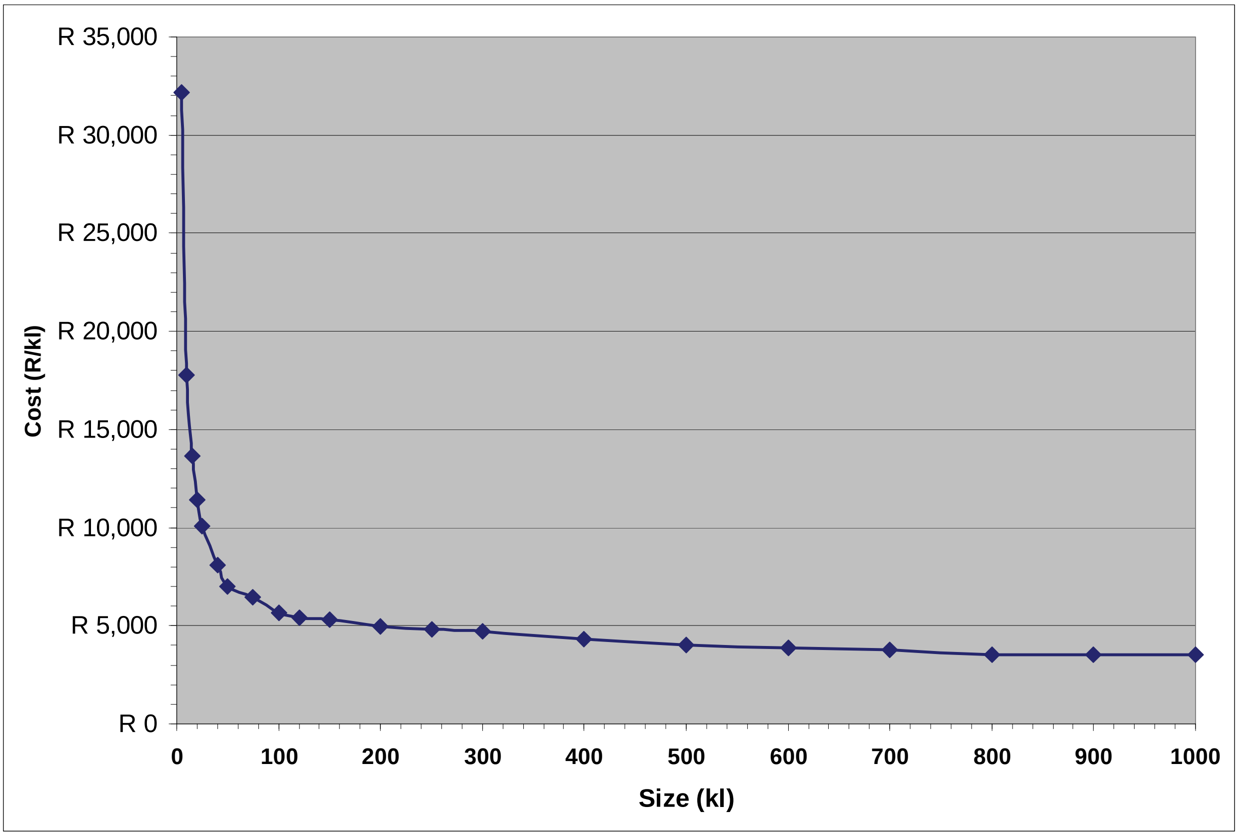

Figure 1: Current costs for reinforced concrete reservoirs (Partners in Development, 2010)

To determine whether it is worthwhile choosing a less expensive alternative to reinforced concrete one has to have some idea of how long such an alternative reservoir will last. Plastic, for example, is the cheapest option for small reservoirs (e.g. 5 kℓ rainwater tanks), but realistically it only has a life of about 10 years (with more exposure to ultra violet rays a plastic tank will lose strength sooner, and vice versa). It is in fact cheaper to replace a small plastic reservoir every ten years than to build one similar sized reinforced concrete reservoir that lasts a hundred years, although the regular replacement of the reservoir is wasteful and a nuisance. If the reservoir is to be located on public land where it may be vulnerable to accidental damage or vandalism, plastic is less attractive. Fibreglass has the same advantages and disadvantages as plastic, but is generally more expensive and not necessarily much more durable.

Masonry is often used for small reservoirs as it is a widely understood building method and no complicated and expensive shuttering system is required. However, it has the inherent disadvantage of having a high number of joints, each of which constitutes a potential leakage path. The tensile strength is provided by steel built into the masonry, and if this steel is not adequately protected from water it will eventually corrode and fail. For this reason masonry reservoirs are not very durable, with 20 years being considered a good lifespan.

Any form of steel reservoir is vulnerable to corrosion. There are various options for protecting steel from corrosion such as hot-dip galvanising, plastic coating and membrane lining. Depending on the sophistication of these systems the lifespan of a steel reservoir is quite variable, from as little as five years but with claims of up to 65 years being made. The key to the longevity of the reservoir is the integrity of the lining and corrosion protection coating, and this must be protected from damage and must be timeously repaired or replaced if it is damaged. Typically to get a life span of 20 to 50 years the full life cycle cost of a steel reservoir will not be very different to reinforced concrete. The main advantage of steel reservoirs is the ease and speed with which they can be erected, and their suitability for erection on elevated stands.

Ferrocement has been in use for reservoir construction since the 1950s and is reasonably well known internationally as an appropriate technology for small reservoir construction (Reed et al 1993 and Watt, 1978). Ferrocement is in essence steel mesh reinforced cement rich mortar, and ferrocement structures are distinct from reinforced concrete in that they are thin shell structures. Ferrocement reservoir walls and roofs are typically between 40 and 60 mm in thickness, compared to the 150 to 250 which is more typical of reinforced concrete reservoirs. For this reason ferrocement reservoirs can be built for significantly less than the cost of reinforced concrete reservoirs (the exact saving will depend on the circumstances and the context, but a 40% to 50% cost saving is fairly easily achieved). Usually with any cost saving there is a catch or a caveat. The question is, with ferrocement being so much cheaper than reinforced concrete, will it last as long, and if not as long, then how long will it last?

The principle objective of this study has been to answer that question. A secondary objective has been to compile guidelines for the design and construction of ferrocement reservoirs.

1.1 What is ferrocement?

Ferrocement is the name given to a composite material made from a high strength mortar cement mix reinforced with steel mesh and wire. It is distinct from reinforced concrete in the following ways:

- i) no stone aggregate is used in the mix
- ii) the steel reinforcing consists of small diameter wires and mesh closely spaced, rather than larger diameter bars spaced further apart
- iii) minimum thicknesses of ferrocement elements are small, from as little as 10 mm
- iv) ferrocement is not poured between and moulded by shutters, but is worked by hand into the steel mesh reinforcement
- v) due to this construction method ferrocement can be moulded into any shape

In its pure form ferrocement has a steel content of over 5%, which would be very high for reinforced concrete. The total steel content, however, is not necessarily higher because ferrocement structures are thin wall structures (American Concrete Institute, 1987). Ferrocement water tanks tend to deviate from pure ferrocement in that the walls are somewhat thicker and the steel content is not as high as 5% (Watt, 1978).

1.2 A short history of ferrocement

The first recorded use of steel in combination with cement is attributed to the Frenchman Joseph Louis Lambot, who in 1847 took out a patent on a rowing boat made with cement mortar reinforced with steel bars and wire mesh (American Concrete Institute, 1987). Before long the new material had been used for items as diverse as flower pots and guard rails. The French called this new material "ferciment". Soon after this an Englishman named Wilkinson realised the value of steel reinforced concrete beams in buildings, beginning a revolution in building design on which all modern structural concrete engineering is based. With this being the main commercial application and with steel rods being more easily available than steel mesh, the use of steel mesh in reinforcing thin shell structures was for a period forgotten.

Ferrocement, as distinct from reinforced concrete, was rediscovered by the Italian Pier Luigi Nervi in the 1940s. He realised that the material was distinct from ordinary reinforced concrete in its versatility, its relative flexibility and elasticity, and its toughness. He successfully used ferrocement for large span moulded thin shell roof construction and for ocean-going yachts, spawning the "concrete yacht" boat industry. [Advocates of ferrocement boats point out that unlike steel boats they do not rust, unlike timber they are immune to boring insects and unlike fibreglass they do not become brittle from long exposure to ultraviolet light. Their disadvantage is weight].

It was not long before the value of ferrocement in the construction of water retaining structures was realised. With its high cement content and low cement-water ratio, the mortar used in the construction of ferrocement is significantly less permeable than ordinary concrete. Furthermore the thin shell nature of the construction means that less material is used than is the case for a reinforced concrete reservoir of similar size. SB Watt in his 1978 book *Ferrocement Water Tanks and their construction* describes ferrocement water tanks built since the 1950s in the United States, New Zealand, South East Asia and Southern Africa (specifically in Botswana and the then Rhodesia (now Zimbabwe)).

In the 1970s the Asian Institute of Technology (AIT) in Bangkok established the International Ferrocement Information Center (IFIC). For a time this centre published articles and manuals on ferrocement such as Sharma and Gopalaratnam's *Ferrocement Water Tanks* (1980), although it no longer appears to be as active as it was in 80s and 90s. In 1987 the American Concrete Institute published its *State of the Art Report on Ferrocement* (ACI, 1987). The principal author of this report was AE Naaman, who in 2000 published the book *Ferrocement and Laminated Cementitious Composites*, which is now regarded as the standard text on ferrocement as a structural material. In 1993 *Waterlines*, at the time undoubtedly the most widely read Appropriate Technology focused periodical in the world, dedicated its Technical Brief No. 36 to the topic of ferrocement water tanks (Reed et al, 1993).

An internet search on the topic of ferrocement produces a large number of references, although by no means all of these are active. Perhaps the most useful site is that hosted by the Ferrocement Educational Network (FEN) at www.ferrocement.net. This site links active ferrocement technologists from all over the world and its discussion pages are current.

1.3 Ferrocement reservoirs in South Africa

While ferrocement yachts were being built in South Africa from the 1960s, another two decades passed before the introduction of ferrocement for water tanks or reservoirs. Initially its use was restricted to small reservoirs (typically 5 kℓ) used in conjunction with spring protections and rain water harvesting by NGOs engaged with rural development work such as the Valley Trust and World Vision. The first known use of ferrocement for a larger reservoir in South Africa was by the CSIR (Council for Scientific and Industrial

Research) who used it for two 40 kℓ reservoirs at KwaHlophe in the Ndwedwe District of KwaZulu-Natal in 1991. In 1992 the CSIR again used ferrocement construction for two 75 kℓ reservoirs at the neighbouring KwaNyuswa Water Project, as well as for a number of smaller reservoirs. In 1993 the CSIR built a number of 100 kℓ ferrocement in Maphumulo, the neighbouring district. The basis for the original CSIR design was provided by the Building Research Institute's Graham Simpson, who had built ferrocement reservoirs up to 100 kℓ in size when he was working in rural water supply in the then Rhodesia (Simpson, 1998). His CSIR colleague David Still adopted this design changing the tent style catenary shaped roof favoured by Simpson to a dome shaped structure. Since the early 90s several hundred ferrocement reservoirs in the size range 5 kℓ to 220 kℓ have been built in South Africa based essentially on the design first used by the CSIR at KwaNyuswa.

Figure 2: 40 kℓ ferrocement reservoir constructed in 1992, photographed during this research in July 2008. KwaNyuswa area, Ndwedwe, Ilembe District Municipality, KwaZulu-Natal, South Africa.

The largest ferrocement reservoirs built to date in South Africa were two 450 kℓ tanks built in 1995 at Osindisweni, some 15 kilometres west of Verulam in KwaZulu-Natal (Figure 3). These were designed by the consultants James Crosswell and Associates and built by Exter Construction. Although the client at the time was the regional bulk water utility Umgeni Water, the reservoir has since 2000 been owned and maintained by eThekwini Water Services (i.e. Durban Metro). The walls and roof are approximately 60 mm in thickness. The reservoirs were inspected in September 2008 and were found to be in excellent condition (Figure 4).

Figure 3: One of the two 450 kℓ ferrocement tanks located at Osindisweni some 15 kilometres west of Verulam.

Figure 4: An inside view of one of the 450 kℓ Osindisweni Reservoirs, photographed in 2008 some 13 years after commissioning. The reservoir is in excellent condition.

South African engineers have not been quick to adopt ferrocement technology. Professional engineers are conservative by nature, not wanting to specify or use materials or systems which have not stood the test of time. Ferrocement may cost only half as much as reinforced concrete, but does that mean it only lasts half as long, or possibly even less? Despite this reluctance, the list of consulting engineers and organisations that have some experience with ferrocement is not insubstantial, including the following:

- BKS
- Bosch and Associates
- CSIR
- Davies Lyn Partners
- eThekwini Water Services
- James Croswell Associates
- Jeffares Green
- Kwezi V3
- Maluti GSM
- MBB
- Mvula Trust
- Peter Glover Associates
- Partners in Development
- Rural Advice Centre
- Umgeni Water
- VelaVKE

Many of the earlier ferrocement reservoirs constructed by the CSIR were built by relatively unskilled community based construction teams, rather than by professional contractors. One of the advantages of this technology is that it is labour intensive and community builders who are familiar with plastering work can be trained quite quickly in its use. However unless the team is led by a core of skilled and experienced workers the end result, although functional, can be rather more agricultural in appearance than might be desirable. The quality of the ferrocement reservoirs built in the early 1990s was therefore mixed, as will be seen in Section 3 below.

In the past fifteen years several hundred ferrocement reservoirs have been constructed for rural water projects in South Africa. Most of these have been located in KwaZulu-Natal, with others in the Eastern Cape, Limpopo Province and Mpumalanga. Sizes have ranged from 5 kℓ to 450 kℓ.

David Still, the CSIR engineer who adopted Simpson's design to build some of the early CSIR reservoirs at KwaNyuswa and Mapumulo, established the Pietermaritzburg based engineering consultancy PID in 1993. PID has since that time gained extensive experience with ferrocement reservoirs and has since 1996 maintained a ferrocement reservoir construction team. While this has never been PID's core work and while PID has not done much in the way of marketing this capability, over the years PID has been subcontracted by numerous consultants and contractors to design and build these reservoirs and has thus become the most experienced design and construction team in South Africa as far as ferrocement water reservoirs are concerned. One of the reasons PID did not market ferrocement more aggressively was a desire to wait and see how the reservoirs would stand up over time. This study has provided the opportunity to shed light on that question.

2. THE CASES FOR AND AGAINST FERROCEMENT RESERVOIRS

Given enough budget and if time is not a constraint, there are few circumstances where reinforced concrete is not the preferred option for water storage. However, as has been pointed out in the introduction above, there is a place for alternative reservoir systems.

In its favour ferrocement offers the following advantages:

- Relatively inexpensive. Ferrocement is still quite costly compared with, say, plastic. However PID has frequently invited contractors to quote on both reinforced concrete and ferrocement reservoirs in the same tender document, and have typically found that the latter come in at 60% of the price, with all other things being equal (i.e. location, pipe work, labour rates, etc., etc.)[1]. [If this saving was invested long term at an interest rate of 4% above inflation, after just 11 years this investment will amount to more than the cost of the ferrocement reservoirs. After 24 years it will amount to more than the cost of the reinforced concrete reservoirs. After 52 years it will amount to more than three times the cost of the reinforced concrete reservoirs.]

- Strong. There is a misconception that ferrocement reservoirs are made of "plaster". Typically the sand:cement mix for ferrocement is 2.5:1 or 3:1 (unlike plaster which is usually 6:1), and the sand is a mix of plaster sand and concrete sand. The resultant matrix is much stronger than conventional plaster or mortar. One could call it a fine matrix concrete. For example, between February and November 2006 PID took 35 cube samples from a set of ferrocement reservoirs built in the Eastern Cape near Matatiele. The median crushing strength of these samples was 30.2 MPa, with a standard deviation of 6.8 MPa, which is comparable to good quality concrete.

- Durable. As has been shown in the introduction, ferrocement is a form of reinforced concrete. The difference is in the cement content (higher), steel content (greater by %), aggregates (finer)

[1] **Cost comparison of ferrocement and reinforced concrete reservoirs extracted from PID tenders**

Project	Tender closure date	Reservoir size (kℓ)	Ferrocement reservoir cost	RC reservoir cost	FC/RC %
Nokweja	28/10/2004	5	R12 373.34	R20 000.00	62
		10	R16 928.56	R28 110.56	60
		15	R18 436.69	R33 871.31	54
		30	R46 718.28	R55 789.13	84
		70	R58 259.87	R97 657.05	60
		120	R79 436.43	R133 946.32	59
Ezibayeni	23/02/2007	5	R19 602.98	R47 638.86	41
		10	R26 184.65	R52 575.12	50
		15	R29 288.77	R58 541.96	50
		25	R50 819.97	R74 651.90	68
		40	R72 459.04	R100 585.74	72
		120	R134 716.09	R190 023.22	71
Isihlangwini	25/01/2008	5	R35 431.95	R56 285.24	63
				Average cost ratio	**61**
				Median	**60**

Note that all prices given in this table exclude VAT as well as any earthworks and pipework. Note that these costs are inclusive of site establishment costs, supervision costs, and overheads – i.e. they are not just materials and labour costs, which do not adequately reflect the differences in supervision and management costs.

and shell thickness (thinner). Does this have similar durability to conventional reinforced concrete as we know it? That is the question this study seeks to answer.

- Labour intensive. Ferrocement is typically mixed and applied by hand, which makes it a labour intensive process. In countries with scarce and expensive labour, this would be a disadvantage. However, where unemployment is high, where intensive community involvement in the construction is desirable, and where labour is not expensive, this is in fact an advantage.

- Less capital intensive. To build reinforced concrete reservoirs requires expensive steel shuttering and typically machine mixed concrete, both of which require capital investment. Ferrocement requires neither. In fact the quality of hand mixed mortar is better than machine mixed mortar. [With the latter it is found that workers too easily overestimate the water requirement.]

- No shutters necessary. Ferrocement structures can be built with or without shutters. If no shutters are used several layers of a finer mesh are bound together to create a surface into which the first layer of mortar is worked.

- Can be built in remote locations. Due to the construction methodology it is easier to build a ferrocement reservoir in a remote and hard to access location than it is to build a reinforced concrete reservoir in a similar location.

- Relatively quick to construct. The complete construction time for a ferrocement reservoir is typically relatively short compared with reinforced concrete (although not as quick as modular steel reservoirs). The typical time for the construction of the smaller reservoirs (5 to 15 kℓ) is 7 days, although with site preparation and roof curing time the full construction period is three weeks. For the larger reservoirs (20 to 200 kℓ) the full construction time is 4 to 5 weeks.

The disadvantages of ferrocement are mainly aesthetic:

- Not as professional looking as reinforced concrete. Ferrocement is built by hand. It does not have the very even off shutter finish that one gets with concrete. Those used to the appearance of concrete who are not familiar with ferrocement can find this disconcerting.

- Micro-cracking. Ferrocement is more flexible than concrete due to its thin shell structure. This leads to micro-cracking in the surface of the walls. In some cases these cracks weep for a period until they seal by calcification. While this is only an aesthetic problem, it affects the confidence with which the reservoirs are regarded. The solution is very simple. If the reservoirs are painted after they have sealed the cracks are no longer an aesthetic problem. See Section 3 below for a discussion on microcracking with several illustrations.

- Seepage from the base-wall joint. Many ferrocement reservoirs are built with a very simple base-wall joint, and in the larger diameter reservoirs there are sometimes patches where this joint seeps or weeps. Reinforced concrete reservoirs typically use a kicker which effectively raises the base wall joint above floor level, and then this joint is carefully bandaged and sealed. Builders of larger ferrocement reservoirs are advised to use bandages and sealants on their base-wall joints.

- Concern over durability. There is a perception that ferrocement reservoirs being cheaper, thinner walled and "made of plaster" will not last. This study examines whether that perception has any foundation.

One of the questions that needed to be answered in this study was whether ferrocement reservoirs required more maintenance than reinforced concrete reservoirs. For most Water Services Authorities and Providers in the rural areas of South Africa this question would be difficult if not impossible to answer, for the simple reason that planned maintenance of reservoirs is not practiced. eThekwini Water Services, however, did provide an opportunity to gain some insight, as they have a comprehensive reservoir maintenance program and amongst the 400 reservoirs in their system they do have 10 ferrocement reservoirs, all built during the period 1995-1996. These reservoirs had prior to the interview all been inspected by PID and were found to be well maintained and in excellent condition. When interviewed the responsible senior operations managers were not aware that they even had any ferrocement reservoirs, which is evidence that these reservoirs had come to be regarded as simply a variation of concrete reservoir and were not in any way regarded as "problem" reservoirs. (Mervin Govender and Neil Gordon, September 2009).

3. PERFORMANCE OF FERROCEMENT RESERVOIRS IN THE FIELD

Large ferrocement reservoirs have been in use in South Africa for up to 20 years. Based as they are on a thin shell design they are cheaper to build than standard reinforced concrete. What most engineers want to know, however, is whether they can be trusted to stand up in the long term. The principle objective of this study was to carry out field inspections of ferrocement reservoirs in practice to see how they had weathered in the field. A total of 41 reservoirs were visited at eight different locations and they were checked for signs of spalling, cracking or leakage. Any defects observed were recorded.

Figure 5 below shows the approximate location of each of these projects.

Figure 5: Location of water supply projects where ferrocement reservoirs were inspected.

The field inspection checklist included the following questions:

a) Is the reservoir in use?
b) Are there any visible leaks from the reservoir, i.e. through the base, base-wall joint, walls and wall-roof joint?

c) Are there any rust stains visible on the outside of the reservoir?
d) How evident is the micro-cracking in the reservoir walls? This intensity was rated on a scale of low, medium or high.
e) Is there any spalling evident on both the outside and inside of the reservoir?
f) Is there any visible erosion around the reservoir and scour headwall?

Details of the eight projects as well as the findings of the field investigations are discussed below.

3.1 Montebello Community Water Supply Scheme

3.1.1 Project details

The Montebello Community Water Supply Scheme (CWSS) is situated between Wartburg and Tongaat off the R614 (approximately 30 km east of Wartburg). The project falls within the boundaries of the Ilembe District Municipality and construction was completed in 1997.

Table 1 below shows the sizes and numbers of all ferrocement reservoirs inspected on the Montebello CWSS.

Table 1: Ferrocement reservoirs inspected on the Montebello CWSS

Size (kℓ)	Quantity	Location
5	6	All along main road below Montebello hospital
25	1	At treatment works
100	1	At treatment works

3.1.2 Field investigations

Eight reservoirs in total were investigated during a site inspection carried out on the 15 July 2008. Six out of the eight reservoirs inspected were 5 kℓ in size. These reservoirs were all built in pairs alongside the main road leading down from the hospital. Two sets of reservoirs showed no signs of leaks while the one set was leaking from the hatch and wall/roof joint. This had been caused by the float valves not sealing. The intensity of microcracking on all of these reservoirs was low. Figure 6 shows two of the 5 kℓ reservoirs surveyed.

The other two reservoirs inspected were 25 kℓ and 100 kℓ in size respectively. The 25 kℓ reservoir had a few minor areas of seepage through the base/wall joint and through the wall itself while the micro-crack intensity could be classed as high. The 100 kℓ reservoir only exhibited medium micro-crack intensity but had minor seepage around approximately 20% of the base/wall joint. The 100 kℓ reservoir also had approximately 0.5 m^2 of spalling on a portion of the roof. Figure 7 shows the 100 kℓ reservoir while Figure 8 provides a close up view of the spalling occurring on the roof of this reservoir.

Figure 6: Two 5 kℓ ferrocement reservoirs constructed on the Montebello CWSS (built and used since 1997 – inspected July 2008). Simple concrete covers were used for the access hatches for these early reservoirs. Although inexpensive, these covers are heavy and awkward to use.

Figure 7: A 100 kℓ ferrocement reservoir constructed on the Montebello CWSS (built and used since 1997 – inspected July 2008)

3.2 Emayelisweni Community Water Supply Scheme

3.2.1 Project details

The Emayelisweni Community Water Supply Scheme (CWSS) is situated between Wartburg and Tongaat off the R614 (approximately 35 km east of Wartburg) and borders on the Montebello CWSS. The project falls within the boundaries of the Ilembe District Municipality and construction was completed in 1998.

Table 2 below shows the sizes and number of all ferrocement reservoirs inspected on the Emayelisweni CWSS.

Table 2. Ferrocement reservoirs inspected on the Emayelisweni CWSS

Size (kℓ)	Quantity	Location
15	3	All located along hillside near Shembe mountain

3.2.2 Field investigations

Three reservoirs were investigated during a site inspection carried out on the 15 July 2008. All three reservoirs were 15 kℓ in size. These reservoirs displayed seepage halfway up the walls (see Figure 8). On the first reservoir, the leak extended around approximately $^{2}/_{3}$ of the reservoir circumference. On the second and third reservoirs, the leak extended around approximately $^{1}/_{4}$ and $^{1}/_{3}$ respectively of the circumferences. The first reservoir also had seepage around approximately 10% of the base/wall joint. No spalling was evident on these reservoirs but there were some definite rust stains on the walls from the seepage zone. Figure 8 shows the first of the three 15 kℓ reservoirs inspected.

Figure 8: A 15 kℓ ferrocement reservoir constructed on the Emayelisweni CWSS (built 1998, inspected 2008) showing a horizontal crack approximately midway up the wall.

Of the 41 reservoirs inspected in the course of this project these were the only ones that displayed this horizontal crack. The design used for the 5 to 15 kℓ reservoirs uses only horizontal reinforcing as the main stresses in the wall are hoop stresses. Fowl netting is used in conjunction with this reinforcement to bind the mortar and distribute stresses. Unlike the design used for larger (20 kℓ and up) reservoirs there was no high tensile vertically oriented reinforcement. The horizontal cracks in these reservoirs would have been prevented by the inclusion of vertical steel, and although this same failure was not observed on other projects where reservoirs of the same size were inspected, this is an indication that the design of ferrocement reservoirs of this size should include vertical steel as a precaution. This failure mode has never been observed in 5 kℓ reservoirs which are built with a similar design, presumably because the stresses are significantly lower.

3.3 Esigedleni / KwaHlophe Community Water Supply Scheme

3.3.1 Project details

The Esigedleni / KwaHlophe Community Water Supply Scheme (CWSS) is situated between Wartburg and Tongaat off the R614 (approximately 50 km east of Wartburg). The project falls within the boundaries of the Ilembe District Municipality and construction was carried out in 2003 and 2004. The implementing agent for this project was Aquamanzi Developments, the KZN BOTT consortium (with Jeffares Green as engineers and WBHO as main contractors). PID was subcontracted to build the ferrocement reservoirs. Two of the reservoirs on the project were, however, built during an earlier phase in 1992 with the CSIR providing the engineering services.

Table 3 below shows the sizes and number of all ferrocement reservoirs inspected on the Esigedleni / KwaHlophe CWSS.

Table 3: Ferrocement reservoirs inspected on the Esigedleni / KwaHlophe CWSS

Size (kℓ)	Quantity	Location
40	1	Located above the main dirt road running through the KwaHlophe area
75	1	Located alongside the 200 kℓ reservoir in the KwaHlophe area
125	2	Located alongside the main dirt road running through the Esigedleni area
150	2	Located alongside the main dirt road running through the KwaHlophe area
200	1	Located alongside the main dirt road running through the KwaHlophe area

3.3.2 Field investigations

Seven reservoirs were investigated during a site inspection carried out on the 15 July 2008. The first two reservoirs inspected were in the Esigedleni portion of the project and were 125 kℓ in size. These reservoirs displayed some minor seepage through a few hairline radial cracks in the base, around approximately 10% of the base/wall joints and on a few small patches on the walls. The micro-crack intensity on these reservoirs was classed as low. No spalling was evident. Figure 9 shows one of the 125 kℓ reservoirs surveyed. There was some evidence of rust streaking on the walls resulting from seepage at the roof wall joint. The design does not provide for water to rise above this joint and this only occurs if the overflow pipe level is set too high.

Figure 9: A 125 kℓ ferrocement reservoir constructed on the Esigedleni CWSS in 2003/2004 (inspected 2008). Rust streaks on walls are the result of the overflow pipe level being set too high – this level should be set below the roof joint.

The two 150 kℓ reservoirs (located in the KwaHlophe area) were found to be in relatively good condition. There was evidence of minor seepage around 25% of the base/wall joint on both reservoirs as well as a small amount of seepage on the one reservoir where the inlet pipe passes through the wall. The micro-crack intensity was low. The 200 kℓ reservoir (located in the KwaHlophe area) was also found to be in good condition. There were a few hairline radial cracks in the base which do not appear to be serious but should be monitored. There was also some minor seepage through the base/wall joint as well as one small wet spot on the wall. The micro-crack intensity was also low. Figure 10 shows the 200 kℓ reservoir.

Figure 10: A 200 kℓ ferrocement reservoir constructed on the KwaHlophe CWSS in 2004 (inspected 2008)

The 40 kℓ and 75 kℓ reservoirs inspected formed part of the original KwaNyuswa CWSS that was constructed in 1992. These reservoirs were both in remarkably good condition with only a small seep through a couple of small radial cracks in the base of the 40 kℓ reservoir and no visible leaks at all on the 75 kℓ reservoir. Figure 11 shows the 40 kℓ reservoir.

Figure 11: A 40 kℓ ferrocement reservoir constructed on the KwaHlophe CWSS (built 1992, inspected 2008)

3.4 Ncwadi Community Water Supply Scheme

3.4.1 Project details

The Ncwadi Community Water Supply Scheme (CWSS) is situated between Boston and Bulwer off the R617 (approximately 60 km west of Pietermaritzburg). The project falls within the boundaries of the Sisonke District Municipality and construction was completed in 2003. The design engineers for this project were PID and the main contractor was Afropipelines.

Table 4 below shows the sizes and number of all ferrocement reservoirs inspected on the Ncwadi CWSS.

Table 4: Ferrocement reservoirs inspected on the Ncwadi CWSS

Size (kℓ)	Quantity	Location
15	3	Situated in the areas of Gudlintaba and Ezibovini

3.4.2 Field investigations

Three reservoirs were investigated during a site inspection carried out on the 31 July 2008. All three reservoirs were 15 kℓ in size and were constructed in 2002/2003. Unfortunately, none of the reservoirs had water in them at the time of inspection due to a problem with the pumps supplying the scheme. The micro-crack intensity of the first reservoir was of a medium intensity but the reservoir had not had water in it for about a year as there was a problem with the line supplying this reservoir. The other two reservoirs are located next to one another and according to the scheme operator, one reservoir has a slight leak from the base/wall joint. These reservoirs did not display the same fault as the 15 kℓ reservoirs on the Emayelisweni CWSS (see Figure 8 above). Figure 12 shows the first 15 kℓ reservoir inspected.

Figure 12: A 15 kℓ ferrocement reservoir constructed on the Ncwadi CWSS (constructed 2003, inspected 2008)

3.5 Makoba Community Water Supply Scheme

3.5.1 Project details

The Makoba Community Water Supply Scheme (CWSS) is situated between Swartberg and Matatiele (approximately 25 km north-east of Matatiele) and along the road to the Ramatseliso's Gate border post into Lesotho. The project falls within the boundaries of the Alfred Nzo District Municipality and construction was completed in 2004. The consultants on this project were Maluti GSM and the main contractor was Icon construction.

Table 5 below shows the sizes and number of all ferrocement reservoirs inspected on the Makoba CWSS.

Table 5: Ferrocement reservoirs inspected on the Makoba CWSS

Size (kℓ)	Quantity	Location
5	1	Below the dirt road en route to the border post into Lesotho at Ramatseliso's Gate
10	1	Situated in the Rochdale area
13	1	Situated in the Mahangu area
22	4	Situated in the Manderstone, Mahangu, New Rash and Shenxa West areas
39	1	Situated in the Lubaleko area
47	2	Situated in the Mposhongweni and Afsondering areas
71	2	Situated in the Shenxa and Vikunduku areas

3.5.2 Field investigations

Twelve reservoirs were investigated during a site inspection carried out on the 31st July and the 1st August 2008. The reservoirs inspected were in good condition with no leaks found on any of them. A few of the reservoirs displayed a high intensity of micro-cracking although these cracks had all sealed. The 10 kℓ reservoir at Rochdale had a few small sections on the wall and roof where the outer coat of plaster was loose in places. Figure 13 shows this reservoir.

Figure 13: A 10 kℓ ferrocement reservoir constructed on the Makoba CWSS (built 2004, inspected 2008)

Figure 14 and Figure 15 are of the 5 kℓ reservoir and the 22 kℓ reservoirs at Manderstone.

Figure 14: A 5 kℓ ferrocement reservoir constructed on the Makoba CWSS (built 2004, inspected 2008)

Figure 15: A 22 kℓ ferrocement reservoir constructed on the Makoba CWSS (built 2004, inspected 2008)

3.6 Ogunjini Water Supply Scheme

3.6.1 Project details

The Ogunjini Water Supply Scheme was built at Osindisweni, some 13 km west of Verulam, in 1995. The implementing agent (the employer body) was Umgeni Water, the regional water utility. The consultant was James Croswell Associates and the contractor was Exter Construction. The project made use of ferrocement for two 450 kℓ reservoirs and one 250 kℓ reservoir. The two 450 kℓ reservoirs[2] are the largest ferrocement reservoirs built to date in South Africa and are therefore of special interest. These reservoirs are also of interest because, unlike the reservoirs shown in sections 3.1 to 3.5 above which were built mostly by relatively unskilled work teams, the Ogunjini reservoirs were built under professional supervision. They are also of interest because since 2000 they have been owned by eThekwini Water Services (Durban) who almost uniquely in KwaZulu-Natal do have a regular reservoir maintenance programme for all their reservoirs. Interestingly enough when the senior managers responsible for the upkeep of these reservoirs were interviewed it was established that they were unaware that these reservoirs were not reinforced concrete reservoirs. They were certainly not regarded as problem reservoirs.

Table 6 below shows the sizes and number of all ferrocement reservoirs inspected at Ogunjini on 23 September 2008.

Table 6: Ferrocement reservoirs inspected at Ogunjini

Size (kℓ)	Quantity	Location
450	1	Next to district road, 150 metres west of Thuthukani Trading Store
250	1	Next to district road, 2 km west of Thuthukani Trading Store
450	1	Just north of Osindisweni Hospital

[2] 450 kℓ is the nominal size and is the water capacity excluding operational freeboard. The reservoir walls are 14.8 metres in diameter and 3 metres in height, which makes a volume up to the roof joint of 516 kℓ.

3.6.2 Field investigations

Figures 16 and 17 show one of the 450 kℓ reservoirs and the 250 kℓ reservoir at Ogunjini. As can be seen the reservoirs have been recently painted, which is typical of all the 400 reservoirs owned and maintained by eThekwini Water Services[3]. They were built without the use of any shutters. The steel and mesh reinforcing was stiff enough to create the shape and the surface into which the first mortar layer was worked (Figure 18). These reservoirs are closer to true ferrocement than the other examples shown in this report, with wall and roof thicknesses as thin as 50 mm (Figure 19) and cover to the steel of 2 to 5 mm (Figure 23). While the 450 kℓ reservoirs were painted with ordinary paint, it was noted that the 250 kℓ reservoir has at some point in its life been coated with a membrane type paint (Figure 20), possibly due to concern about the normal microcracking that is seen with this technology. Some evidence of well sealed microcracks was observed under the paint of the other reservoirs (Figure 21). These figures illustrate the difference that a coat of paint makes to the aesthetics of these reservoirs. The inside of the reservoirs shows no sign of deterioration on either walls or the roof. One very minor seep was observed on the wall floor joint of just one of these three reservoirs (Figure 22).

Figure 16: One of the two 450 kℓ reservoirs built for the Ogunjini Water Project at Osindisweni, west of Verulam. These reservoirs were built in 1995 and inspected in September 2008.

Figure 17: The 250 kℓ reservoir built at Ogunjini in 1995 (inspected 2008). These reservoirs were designed by James Crosswell Associates and built by Exter Construction.

[3] All of eThekwini's smaller reservoirs are re-painted every one to two years routinely. Apart from the aesthetic value of this exercise, the yellow painted roofs make the reservoirs easy to find from the air, and from the ground as well.

Figure 18: A close-up inspection of the wall gives an indication of the fine mesh into which the mortar has been worked. These reservoirs were built without using any shuttering.

Figure 19: Ferrocement is a thin shell technology. The mesh and the mortar form a composite material. The roof of this reservoir, which has a 14.8 metre diameter, is 50 mm thick.

Figure 20: The 250 kℓ reservoir appears to have been painted with a membrane type paint at some point in its life. This may have been in reaction to weeping or micro-cracking. If there was any seepage early on in the reservoir's life, no evidence of this could be seen when it was inspected in September 2008.

Figure 21: This close-up photograph of a section of wall on one of the 450 kℓ reservoirs shows evidence of microcracking which has sealed. The painting of the reservoir makes this hardly noticeable even from a short distance.

Figure 22: One of the 450 kℓ reservoirs had just one small seepage patch at a point on the base-wall joint. The others had none at all.

Figure 23: The Ogunjini reservoirs are more pure ferrocement than the other reservoirs shown in this report. One of the characteristics of true ferrocement is that the cover to the steel is as little as 2 to 5 mm, as is revealed by this small spall. The philosophy behind the very thin cover is that the full thickness of the wall is a mortar and steel composite which is flexible and strong. Less cover is needed to protect the steel from corrosion as the mortar matrix is cement rich and fine grained compared with conventional concrete. Galvanised mesh is recommended for the outer layers most likely to be at risk of corrosion.

Figure 24: A close-up of the underside of the roof of the second 450 kℓ reservoir shows that some patching has taken place.

3.7 Fredville Water Supply Scheme

3.7.1 Project details

The Fredville Water Project was constructed in 1996 in the uMngeni valley approximately mid-way between Durban and Pietermaritzburg. The area is characterised by steep slopes and the reservoirs serve as break pressure tanks and distributed storage. The implementing agent for this project was Umgeni Water and the consulting engineers were BKS. PID were subcontracted to design and build the reservoirs. During the field visit three of these reservoirs were visited.

3.7.2 Field investigations

Table 7 below shows the sizes and number of all ferrocement reservoirs inspected at Fredville.

Table 7: Ferrocement reservoirs inspected at Fredville

Size (kℓ)	Quantity	Location
15	2	Between Duzi Bridge and Foley Marianney Bridge
50	1	Overlooking Mamba Gorge

Three reservoirs were investigated during a site inspection carried out on 23rd September 2008, one 50 kℓ reservoir and two 15 kℓ reservoirs (Figures 25 and 26). The reservoirs inspected were in very good condition. Two had small wet patches at one or two places on the wall floor joint (Figure 27), but there was no evidence of any seepage through the walls on any of the reservoirs. These reservoirs have been owned and maintained by eThekwini Water Services since 2000 and they are regularly painted bright yellow to make them easier to find.

Figure 25: 50 kℓ reservoir at Fredville built in 1996 and inspected in September 2008. The reservoir is owned and maintained by eThekwini Water Services. Reservoirs in remote locations like this are painted yellow to make them easier to locate.

Figure 26: Two 15 kℓ reservoirs built at Fredville in 1996.

Figure 27: Minor seepage observed on approximately 10% of the base-wall joint of the 50 kℓ reservoir. One of the 15 kℓ reservoirs inspected had a similar seep over 5% of this joint, while the other had none.

Figure 28: Inside view of portion of 50 kℓ reservoir.

3.8 Trial of ferrocement to rehabilitate or protect other types of reservoirs

Given its unique construction methodology, ferrocement is useful for the protection and/or rehabilitation of other types of reservoir, for example:

- When galvanised corrugated iron reservoirs rust through (this typically takes less than 10 years) they can be given a new lease of life by wrapping them with wire and mesh and plastering the outside with two layers of a cement rich plaster-river sand mix, thus creating a form of ferrocement reservoir with permanent shuttering on the inside. For best durability, a plaster layer should be added to the inside of the old steel tank as well, otherwise the reinforcing will ultimately rust through.

- If vandalism is a concern, plastic tanks can be wrapped in mesh and plastered.

- If concrete reservoirs are under-reinforced and fail[4], with vertical cracks opening up around the reservoir, they can be rehabilitated by simply scabbling the outside of the reservoir to create a key, wrapping the reservoir in mesh and high tensile wire (well tensioned up), adding two layers of cement rich plaster-river sand mix, and sealing the inside of the reservoir with a good sealant.

In the Nhlungwane Village in the Msinga District, KwaZulu-Natal Province, three poorly reinforced 30 kℓ concrete reservoirs had cracked so badly that they were not able to hold water at all. In 2003 the reservoirs were rehabilitated by applying a layer of ferrocement on the outside, and a site inspection carried out in 2008 confirmed that the repairs have held up excellently. In this way a concrete reservoir can be repaired and put back into service for a fraction of its replacement cost (see Figure 29).

Figure 29: This was a substandard reinforced concrete reservoir built for a community water supply project at Nhlungwane Village, Msinga District which failed after approximately five years' service. In 2003 this reservoir and two others like it were repaired using ferrocement for a fraction of their replacement cost. Inspected in 2008, the repairs were found to have held up soundly.

[4] While professionally designed and constructed reinforced concrete reservoirs are typically if anything over designed and over reinforced, there are many reinforced concrete reservoirs built on farms which are built on a quite different design philosophy and sometimes this type of reservoir fails due to under reinforcement.

3.9 Discussion of field inspections

The field investigations described above have been summarised below according to certain topics of interest. These analyses focused primarily on the incidence of seepage through the base/wall joint, the incidence of seepage through the walls, the intensity of micro-cracking and the incidence of external and internal spalling. The Nhlungwane ferrocement repair of concrete reservoirs (section 3.8 above) are only discussed where relevant.

3.9.1 Seepage Through the Base/Wall Joint

Seepage through the base/wall joint was the most common problem encountered on the ferrocement reservoirs surveyed. Table 8 below shows the summary of the reservoirs inspected that had at least some seepage from the base/wall joint. The reservoirs have been grouped according to their respective projects. Note that the three reservoirs at Ncwadi which did not have water in them at the time of the inspection are not included in this analysis.

Table 8: Seepage through the base/wall joint according to project

Project	Total no. inspected	No. with seepage	% of total with seepage
Montebello	8	2	25%
Emayelisweni	3	3	100%
Esigedleni/KwaHlophe	7	5	71%
Makoba	11	0	0%
Ogunjini	3	1	33%
Fredville	3	0	0%
Totals	**35**	**11**	**31%**

The incidence of seepage was also analysed according to reservoir size. The results of this are shown in Table 9 below.

Table 9: Seepage through the base/wall joint according to reservoir size

Reservoir size (kℓ)	Total no. inspected	No. with seepage	% of total with seepage
5 to 15	14	3	21%
22 to 75	12	1	8%
100 to 450	9	7	78%
Totals	**35**	**11**	**31%**

From Table 9, it would appear that the larger reservoirs, i.e. over 100 kℓ in size, are more likely to have at least one or two patches of seepage through the base/wall joint than are the smaller reservoirs. Given the greatly increased length of this joint on the larger reservoirs, this is not unexpected. During the field investigations, note was made of the percentage of the base/wall joint from which the seepage was occurring. These figures ranged from 4% on the 450 kℓ reservoir at Ogunjini to about 25% on the two 150 kℓ reservoirs on the Esigedleni/KwaHlophe CWSS. Figure 30 shows seepage from the base/wall joint of the 100 kℓ reservoir inspected on the Montebello CWSS.

The picture that emerges of a few wet patches at some or other point of the base wall joint is fairly typical for the ferrocement reservoirs as they were being built in the 90s. The Ogunjini reservoirs, with only one very small wet patch for three large reservoirs, are essentially leak free at the base-wall joint. Although these leaks are small enough that the reservoirs still fall within typical water tightness criteria[5], they are unsightly. Accordingly PID started adding a bandage and sealant to the inside of the base wall joint of its reservoirs from 2004 onwards. It is significant therefore that the Makoba reservoirs, built in 2004, show no evidence of leakage from the base wall joint.

Figure 30: Seepage from the base/wall joint of a 100 kℓ ferrocement reservoir constructed on the Montebello CWSS in 1997. A few patches like this around the base of the larger reservoirs were fairly typical of the earlier large ferrocement reservoirs built in South Africa using the CSIR design. Subsequently PID has specified the addition of a bandage and sealant to his joint, and this successfully prevents the seepage, which is more unsightly than significant.

3.9.2 Seepage Through the Wall

At least one patch of seepage through the wall was encountered on nine of the ferrocement reservoirs surveyed. Table 10 below shows the percentage of the reservoirs observed that had at least some seepage through the wall. The reservoirs have been grouped according to their respective projects. Note that any reservoirs that had no water or else only a small amount of water in them at the time of the inspection were not included in this analysis.

Table 10: Seepage through the wall according to project

Project	Total no. inspected	No. with seepage	% of total with seepage
Montebello	8	1	13%
Emayelisweni	3	3	100%
Esigedleni/KwaHlophe	7	4	57%
Makoba	11	1	9%
Ogunjini	3	0	0%
Fredville	3	0	0%
Nhlungwane	3	0	0%
Totals	**38**	**9**	**24%**

[5] Engineers usually specify that the rate of loss of water from a closed reservoir should not exceed 1 to 2 mm in depth per 24 hours.

The incidence of seepage was also analysed according to reservoir size. The results of this are shown in Table 11 below.

Table 11: Seepage through the wall according to reservoir size

Reservoir size (kℓ)	Total no. inspected	No. with seepage	% of total with seepage
5 to 15	14	4	29%
22 to 75	15	1	7%
100 to 450	9	4	44%
Totals	**38**	**10**	**26%**

From Table 11, it would again appear that the larger reservoirs, i.e. over 100 kℓ in size are more likely to have at least one patch of seepage through the wall than the smaller reservoirs. This can, however, be explained by the increased surface area of the walls on these larger reservoirs.

It should also be noted that (with the exception of the three Emayelisweni reservoirs which were poorly built relative to the others inspected), most of the seepage occurring through the wall on the reservoirs surveyed was very minor and generally only in small areas. For example, Figure 31 shows a small area of seepage through the wall of the 25 kℓ reservoir inspected on the Montebello CWSS.

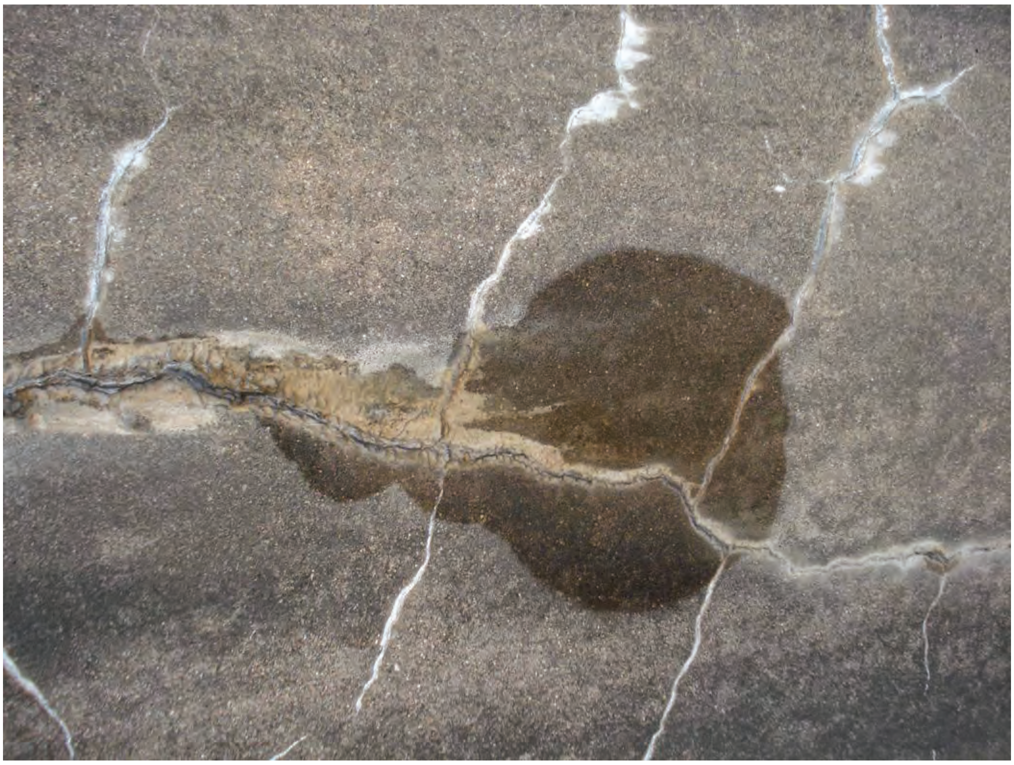

Figure 31: A patch of seepage through the wall of a 25 kℓ ferrocement reservoir constructed on the Montebello CWSS

3.9.3 Intensity of Micro-cracking

Micro-cracking has been observed on all reservoirs included in this study thus far. This cracking is normal for ferrocement as it is a thin shelled flexible structure. Micro-cracking can be observed on the outside of ferrocement reservoirs, but is not seen on the inside. When ferrocement is used for water storage these cracks do self-seal. In order to differentiate between the intensity of micro-cracking on each reservoir, a visual assessment was carried out and the intensity of micro-cracking was rated as low, medium or high. To illustrate this classification, Figures 32, 33 and 34 show reservoirs that were classed as having low, medium and high micro-cracking intensities respectively.

Figure 32: Low intensity micro-cracking on a 13 kℓ ferrocement reservoir constructed on the Makoba CWSS

Figure 33: Medium intensity micro-cracking on a 22 kℓ ferrocement reservoir constructed on the Makoba CWSS

Figure 34: High intensity micro-cracking on a 25 kℓ ferrocement reservoir constructed on the Montebello CWSS

Table 12 below shows the percentage values of the reservoirs in the different micro-cracking categories according to project.

Table 12: Micro-cracking intensity classification according to project

Project	Total no. inspected	Micro-cracking intensity					
		Low	% of total	Medium	% of total	High	% of total
Montebello	8	6	75%	1	12.5%	1	12.5%
Emayelisweni	3	0	0%	0	0%	3	100%
Esigedleni/KwaHlophe	7	7	100%	0	0%	0	0%
Makoba	11	3	25%	6	50%	3	25%
Ogunjini	3	3	100%	0	0%	0	0%
Fredville	3	3	100%	0	0%	0	0%
Nhlungwane	3	3	100%	0	0%	0	0%
Totals	**38**	**25**	**66%**	**7**	**18%**	**7**	**18%**

A point to note from Table 12 is the percentage of reservoirs with micro-cracking of medium and high intensity on the Makoba CWSS, despite all but one of these reservoirs having been completely leak free and the remaining one having had just one minor seepage patch. A possible explanation for this is the extreme changes in temperature that the area is subjected to. Temperatures in summer can reach well above 30°C whereas in winter the area is extremely cold and regularly experiences temperatures of well below 0°C.

It should also be noted that very few micro-cracks observed during the field visits were actually leaking. Thus, while these cracks may look unsightly, they do seal themselves over time. Figure 35 shows a close up view of one of the more substantial micro-cracks showing the natural calcification sealing process that has taken place. It must also be noted that calcification and its associated white streaks and patches are a characteristic of all cement based water retaining structures, from small reservoirs to the largest concrete dam walls.

Figure 35: Close up view of one of the largest micro-cracks observed in the field work showing the calcification that has sealed the crack over time.

3.9.4 Spalling

Spalling occurs when a piece of concrete or mortar flakes away from the structure wall exposing the reinforcing underneath – in fact the flaking is caused by the oxidation of the reinforcement which pushes off the surface material. Spalling is a concern with any reinforced concrete structure and for this reason a minimum cover over the steel is specified. For conventional concrete this cover is typically from 20 to 40 mm, depending on the environmental conditions (the greater the exposure of the structure to salt and moisture, the greater the cover). With its fine aggregate matrix and the high cement content, and also with external layers of mesh often being galvanised, the design approach to cover in the case of ferrocement is quite different, with covers as low as 3 mm being used in some cases, even for ocean going yachts. It was therefore of some interest to observe to what extent spalling had taken place on the ferrocement reservoirs inspected.

A patch of spalling was found on one 100 kℓ reservoir constructed on the Montebello CWSS. The spalling was found on the outside of the roof over an area of approximately 0.5 m^2, i.e. approximately 0.8% of the surface area of the roof. Figure 36 below shows the spalling on this 100 kℓ reservoir. Some patches of spalling were also observed on the underside of the domed roofs where the mesh had not been properly covered. This is not a concern as the mesh in the domed roofs is included as a construction aid and is not structural.

The Ogunjini reservoirs show some evidence of minor spalling, as shown in Figures 23 and 24 above. Unlike the ferrocement reservoirs inspected which had been built according to the CSIR design which was more conservative in terms of the cover on the walls[6] the Ogunjini reservoirs are more true ferrocement in nature with very thin cover. It is therefore notable that they have had very little spalling taken as a whole.

Figure 36: A small section of spalling occurring on the roof of the 100 kℓ reservoir in Figure 6 above. Note that the mesh reinforcing in the domed roof has minimal structural function, and is included mainly as a construction aid. Nevertheless, a proper maintenance programme should see this exposed steel being rust-proofed and patched.

3.9.5 Other Problems

A few other problems observed during the field investigations are worthy of further discussion. Firstly, the problem experienced by the 15 kℓ reservoirs at Emayelisweni, i.e. with a considerable seep occurring from the walls approximately half way between the base and roof joints, indicates that vertical reinforcing should be included on all ferrocement reservoirs larger than 10 kℓ.

A further common problem on the ferrocement reservoirs surveyed is the crack that is often found along the wall/roof joint. Possible reasons for this crack are as follows:

i) There is usually a gap of at least a week between the completion of the walls and the casting of the roof. This means that there is not a strong bond between the roof and the wall.

ii) In the case of the larger roofs there may be a slight movement (a flexion) of the roof that occurs at this joint after the support props are removed at the end of the curing period.

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[6] The CSIR design, which was a combination of Graham Simpson's experience in the then Rhodesia and Frans Diener's method used on the World Vision small ferrocement reservoirs, allowed for two coats of mortar inside the mesh and one outside, with one effectively being in the mesh.

Although this crack is unsightly if it is not patched over with a bagwash or grout, it is not in itself evidence of a structural problem. Radial cracking in the roof, on the other hand, is evidence of inadequate hoop reinforcement in the roof, and this has been observed on some occasions (see the section on roof design, 4.3, below). Figure 37 shows a photograph of the 200 kℓ reservoir on the KwaHlophe CWSS with a more standard crack along the wall/roof joint visible.

Figure 37: The wall/roof joint is often marked by a small crack. This is not a structural joint, nor is it a joint intended to be watertight, as the reservoir overflow level is set approximately 50 mm below this joint. The joint should be grouted or bagwashed after construction.

3.9.6 Summary of findings from field work

The ferrocement reservoirs inspected have been in generally good condition[7], although in some cases minor seepage was observed either through the base/wall joint or the wall itself. Very little spalling was observed. In the case of the reservoirs where a bandage and sealant had been added to the inside of the wall-floor joint, there was no seepage from this joint. The painting of reservoirs as practiced by eThekwini Water Services greatly improves their outward appearance.

It is not possible to speculate whether the reservoirs can be expected to last forty, fifty, a hundred years or longer. All that can be said is that reservoirs which have been in use for 15 to 18 years are in good condition and appear no worse for wear.

[7] Except for the Emayelisweni reservoirs which appear to have been poorly built and which needed more vertical reinforcement.

4. DESIGN CONSIDERATIONS

Apart from the materials and the mix, ferrocement reservoirs differ from concrete reservoirs in that the walls and roof are relatively thin, ranging from as little as 50 mm at the top of the roof to typically 80 mm or so at the base of the walls. The reinforcing is located in the centre of this shell, and not in two layers as in reinforced concrete.

For a detailed treatment of the structural analysis of complex ferrocement structures a text such as *Ferrocement and Laminated Cementitious Composites* by Dr Antoine E. Naaman (Naaman, 2000) should be consulted. Circular reservoirs with cone or dome shaped roofs of diameter up to 12 metres are relatively simple structures, however, and can be designed based on the guidelines in this chapter.

4.1 Base

The total loading that a full ferrocement tank of two metres height exerts on the underlying ground is typically between 24 and 36 kPa. To put this in context the load exerted on the ground by a stationary 100 kg man with his weight spread evenly on both feet (wearing flat soled shoes) is about 20 kPa. In other words, ferrocement tanks do not have to be sited on ground with a high load bearing capacity. Even unconsolidated dune sand, which has a load bearing capacity of less than 20 kPa, is suitable if the tank is built as a continuously reinforced shell on a raft type foundation. What is important is that the ground is level and well drained, and that it is not of uneven load bearing capacity. Typically tanks are located on hilltops or at least on spurs, in which case drainage is generally not a problem. However, if a tank has to be located on poorly drained ground then a grid of subsoil drains[8] should be provided and the top layer of soil should be removed and replaced with stabilized[9] and compacted material. Alternately the top 200 mm of the soil can be replaced with stone aggregate. For most sites, however, all that is necessary is that the topsoil is removed and that the site is levelled. Note that a tank must not be located on a mix of cut and fill, but only on cut. A mix of cut and fill would very likely settle unevenly under load, and this would create large stresses in the tank for which it is not designed.

Although Watt (1978) describes a construction method successfully used in the United States since the 1950s for a 9.7 metre diameter tank with an unreinforced base, most other designs encountered in the literature do show a layer of steel mesh in the base. PID uses a single layer of Ref. 193 weldmesh (5.6 mm high tensile steel bars spaced at 200 mm centres), located 50 mm from the bottom of a 120 mm thick concrete base. Before the concrete base is cast, a 50 mm thick blinding layer of low strength (5:5:1 mix) unreinforced concrete is cast on the levelled ground. The blinding provides a firm support to the spacer blocks or plastic spacers used to ensure that the mesh is located inside the slab and not at the bottom of the slab. It also to some extent seals the ground below the slab, which improves the strength of the concrete (a plastic sheet would have the same effect).

Figures 38 and 39 below show the design detail of a base, and a base under construction.

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[8] One way to build a subsoil drain is to dig a trench, line it with a geotextile fabric (such as Bidim), lay a slotted plastic drainage pipe down the length of the trench, backfill the trench with a free draining materials such as river sand, gravel or stone, then cover the trench with geotextile. If geotextile is not available it can be replaced with graded layers of sand progressing from fine alongside the material to be drained to coarse near the drainage pipe. Subsoil drains are often placed beneath the floors of reinforced concrete reservoirs, particularly below joints in floor panels. In contrast the floors of ferrocement water tanks are typically continuous with no joints.

[9] Soil or sand is stabilised by mixing it with lime or cement and replacing it with moisture and compaction. The percentage lime or cement added depends on the soil and can be determined by a soils laboratory, but 3% to 6% by volume is typical. For a relatively small structure such as ferrocement tank of up to 12 metres diameter one would not need to stabilize to a depth greater than 200 to 300 mm. A geotechnical engineer or a pavement design engineer could advise on how to go about stabilizing the material under a tank if it was considered necessary.

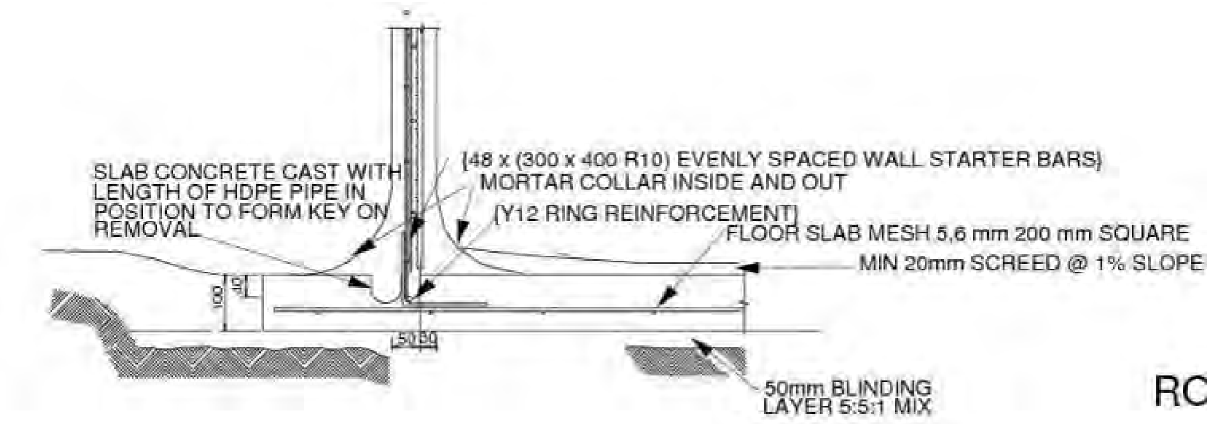

Figure 38: Detail used by PID for base reinforcement and base-wall joint

Although it is unnecessary for the small (5 and 15 kℓ) reservoirs, for the larger reservoirs it is believed to be a good idea to cast a key into the base below the walls, to make the wall-base joint stronger and less prone to leakage (this is shown in Figure 39). A 40 mm HDPe pipe is used to create this key in the concrete (Figure 39). After the concrete has set the HDPe pipe is removed.

Figure 39: Concrete base under construction showing Ref 193 weldmesh laid out in preparation for spacers to be placed on top of blinding. A 40 mm pipe (later removed) creates a key to improve the base-wall joint. A single bar of Y12 high tensile hoop reinforcement strengthens the base under the wall, and 10 mm diameter mild steel bars help to fix the base-wall joint.

4.2 Pipework

Not all texts on ferrocement water tanks adequately deal with the topic of pipework. A properly designed water reservoir must have each of the following:

a) an outlet, being the pipe used to convey water to its intended destination;
b) an inlet, being the pipe used to get water into the reservoir;
c) an overflow, being the pipe used to dispose of surplus water in the reservoir if it fills above its full capacity; and
d) a scour pipe, being the pipe used to drain the reservoir when it is being cleaned.

Except for the inlet, which can be cast through the base or alternately through the top of the wall, it is preferable for the pipes to pass through the base and not through the wall. A scour can only go through the base otherwise it cannot scour the whole of the reservoir's contents. An outlet and overflow can pass through the wall, but this arrangement is more likely to leak around the outside wall of the pipes. A more fail-safe arrangement is to pass these pipes through the base and then under the base in a narrow trench which is backfilled with concrete around the pipes.

Figure 40 below shows a schematic arrangement for all four pipes, and Figure 41 shows the concrete encased scour and outlet sticking out from underneath a tank base.

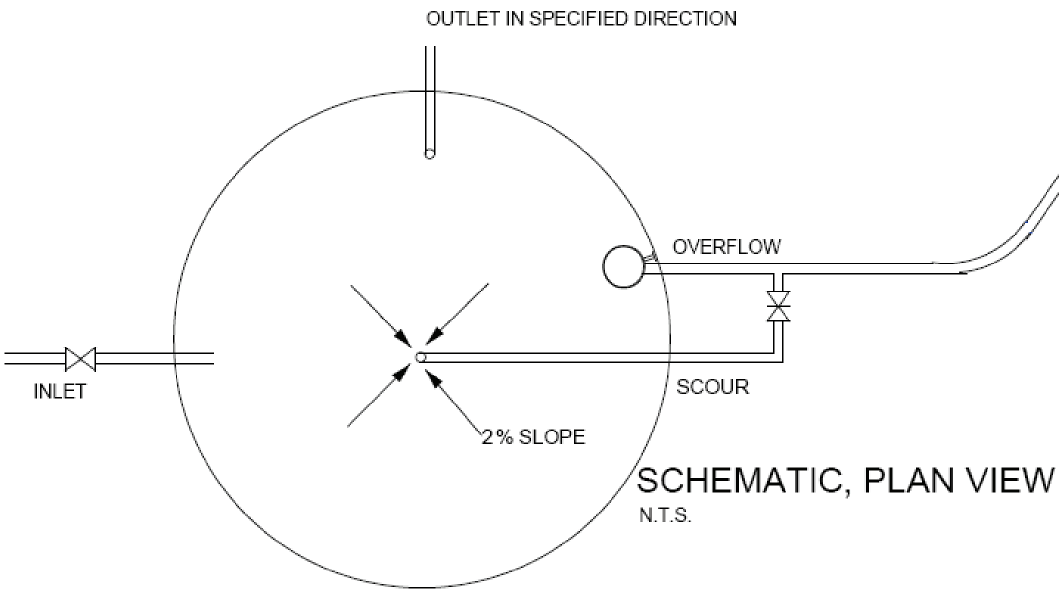

Figure 40: Schematic pipework arrangement

Figure 41: The outlet and scour pipes are cast through and under the base and are encased in concrete. Except for the inlet which is exposed, PVC pipes are used because they will not rust. A bond between the PVC pipes and the concrete is achieved either by rough sanding the outside of the pipe, or by lightly coating it with PVC glue and sand.

A few more points regarding the pipe work must be made:

i) Even galvanised steel rusts over time. Any buried galvanised steel should be protected against rust. The use of bitumen impregnated tape wrapped around the outside of any buried steel pipe (also called “denso wrapping”) will protect the pipe from corrosion.

ii) If the inlet is to be top entry, running up the outside of the reservoir, this should be steel which is vandal proof and also will not deteriorate over time due to ultraviolet rays.

iii) The inlet needs some kind of control valve to stop the tank from over filling. A simple float valve can be used, or a pilot float can be used with a hydraulically operated control valve located in a chamber outside the reservoir. Whichever of these two arrangements are used, the rate of valve closing must not be so quick as to cause excessive surge pressures in the incoming pipeline. The rate of valve closing is determined by the valve’s design or adjustment. A third option for inlet control, only used in association with pumps, is the use of float switches which communicate by cable or telemetry (radio or satellite) with a receiver linked to a pump control switch.

iv) The outlet, scour and overflow are preferably to be cast into the tank base, and for this steel is not a good idea as it eventually rusts even if it does have some form of corrosion protection. Rather use PVC pipe. A bond between the PVC pipes and the concrete is achieved either by rough sanding the outside of the pipe, or by lightly coating it with PVC glue and sand.

v) The outlet must not draw water from the very bottom of the tank, as the floor of any water reservoir collects dirt. Rather the lip of the outlet must be raised a short distance (e.g. 75 mm) above the floor of the reservoir.

vi) The scour must draw water from the very bottom of the tank. It is preferable to apply a mortar screed to the floor of the tank sloped at 1% or more towards the scour outlet. This will make the job of scouring and cleaning the reservoir easier.

vii) The overflow pipe must be set to draw water from above the level of the inlet (otherwise water will overflow before the tank is full) but below the top of the reservoir walls (otherwise the reservoir will leak through the wall-roof joint, which is not a watertight joint).

viii) The sizing of the inlet and outlet can usually be one size down from the pipes to which they are connected. The overflow, however, should be one or two pipe sizes larger than the inlet, as the overflow capacity (which is driven by gravity under a low head) must exceed the inlet capacity (which is usually driven by a much higher head).

ix) The scour and overflow can be joined outside the reservoir, with a valve to isolate the scour. This valve must be situated in a locked chamber if there is any chance that the scour valve might be wilfully or accidentally interfered with.

4.3 Walls

The tensile hoop forces in the walls and at the base of the roof are significant and sufficient steel reinforcing must be provided to contain these forces, or the tank will split apart. The understanding of these forces is the main design consideration in ferrocement tank design.

The CSIR design which was adopted for most of the ferrocement tanks described in this report have circular walls 1.9 metres high. The reasons for the selection of this height are twofold:

1) the shuttering used as a construction aid to shape the walls is made from three standard sheets of corrugated iron, with overlaps between each sheet. This produces a shutter 1.9 metres high.

2) 1.9 metres is the height to which a person can work fairly comfortably without the use of scaffolding. It is therefore a practical height for what is intended to be a labour intensive construction method.

The primary stresses in the walls are hoop stresses generated by water pressure acting to push the walls outwards. If the base wall joint was a sliding joint, the maximum hoop tension would be located at the base of the wall. However, a sliding base-wall joint would be difficult to waterproof, and for this reason the joint is fixed. For the smaller reservoirs the hoop stresses at the base are so low that the shear strength of the mortar joint between the wall and base is sufficient to prevent movement (although note that for all sizes of reservoir the wall is widened at the base to increase the strength of this joint). For the larger reservoirs, above 15 kℓ, L shaped reinforcing bars are tied at regular intervals to the base reinforcing and to the wall reinforcing to reinforce this joint. The bars should be located on the inside of the wall, as the water pressure creates a bending moment which creates a tensile force located on the inside of the base-wall joint. Figure 42 below (after Watt) illustrates the effect of the fixing of the joint on wall stresses.

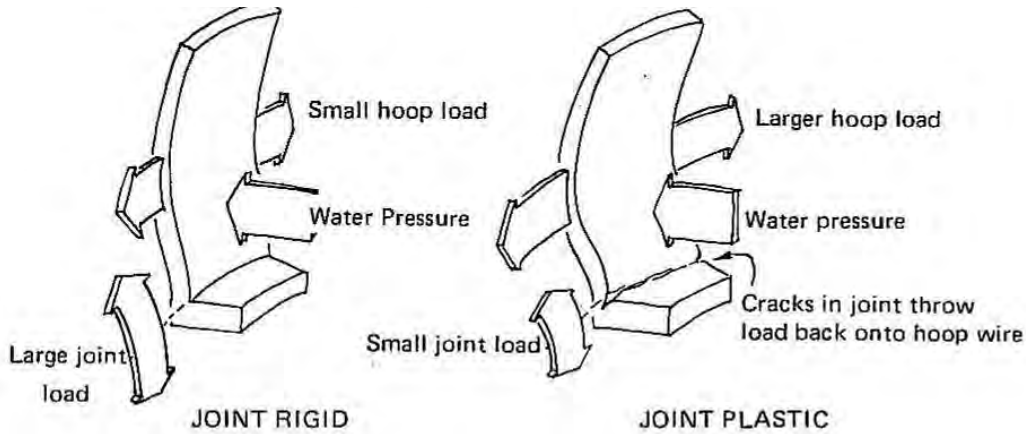

Figure 42: The effect of the fixing of the wall floor joint on wall stresses (after Watt, 1978)

Due to the fixing of the wall-floor joint the maximum hoop stress is found not at the base of the reservoir but part way up the wall[10]. Table 13 shows how hoop stress is calculated for a range of reservoir diameters from 1.9 metres to 10.7 metres. It also shows how a combination of hard drawn (high tensile) wire of diameter 3.15 mm and Ref. 156 weld mesh of diameter 3.55 mm can be used to contain this stress. The assumption is that where the wire is used it will be spaced at 76 mm intervals, which is the spacing of the corrugations of the temporary shuttering. The mesh wires are spaced at 100 mm centres. Both of these types of reinforcing are high tensile, which means that their yield strength is 485 MPa (1 MPa is equivalent to 1 Newton of force per mm^2). A factor of safety of 2 is applied to this strength, which means that the design factor of safety is calculated as 243 MPa divided by the calculated maximum stress. If galvanised wire is used for the reinforcing it must be noted that this is *not* high tensile wire and the yield strength is only 290 MPa. Using galvanised wire to get the equivalent amount of reinforcing the total area of steel provided must be increased by 70%.

The vertical bars in the weld mesh contain the bending stresses that are generated in the wall due to the fixing of the base-wall joint. They also increase the shear strength of the wall in the horizontal plane, which helps to distribute the hoop stresses more evenly over the wall. For these reasons the vertical steel which is gained by the use of the mesh is important. Although the primary stresses in the tank are

[10] As the reservoir diameter increases the h/H ratio, where h is the height above the base of the maximum hoop stress and H is the maximum depth of water, increases. For further details see Watt (1978), who in turn refers to the publication *Reinforced Concrete Reservoirs and Tanks*, by GP Manning, Cement and Concrete Association, 1972.

hoop stresses, a large tank which is reinforced only by horizontally oriented wire may develop cracks in the horizontal plane (see Figure 8 on page 12, for example).

In the case of small ferrocement reservoirs which have no vertical steel to absorb bending stresses, these stresses are catered for partly by the light gauge bird mesh or fowl netting which is included to bind the ferrocement, and partly by the tensile strength of the mortar mix[11].

[11] For more detail on the tensile strength of the mortar mix see page 45.

Table 13: Calculation of hoop stresses and hoop steel provision

NOMINAL VOLUME OF RESERVOIR	4	10	15	25	50	70	125	150
REINFORCING DIAMETER (m)	1.896	2.819	3.488	4.506	6.289	7.435	9.743	10.666
MAXIMUM HEIGHT OF WATER ABOVE BASE (m)	1950	1950	1950	1950	1950	1950	1950	1950
VOLUME OF RESERVOIR (kℓ) WITH 1.7 m WATER DEPTH	4.21	9.73	15.15	25.68	50.81	71.44	123.64	148.5
SAFETY FACTOR ON STEEL	2	2	2	2	2	2	2	2
TENSILE STRENGTH OF MESH OR HARD DRAWN WIRE (MPa)	485	485	485	485	485	485	485	485
ALLOWABLE STRESS ON MESH OR HARD DRAWN WIRE (MPa)	243	243	243	243	243	243	243	243
AREA OF WELD MESH REF 156 per 100 mm (mm^2)	9.9	9.9	9.9	9.9	9.9	9.9	9.9	9.9
NUMBER OF LAYERS OF WELD MESH REF 156	1	1	1	2	2	2	2	2
AREA OF 3.15 mm HARD DRAWN WIRE (mm^2)	7.79	7.79	7.79	7.79	7.79	7.79	7.79	7.79
NUMBER OF CORRUGATION PER 100 mm	1.32	1.32	1.32	1.32	1.32	1.32	1.32	1.32
NUMBER OF HOOPS OF HARD DRAWN WIRE PER CORRUGATION	0	1	1	1	1	2	3	3
MAXIMUM HOOP FORCE ON LAST 100 mm (kN) - without fixity of base joint	1.85	2.75	3.4	4.39	6.13	7.25	9.5	10.4
HEIGHT OF MAX HOOP FORCE AS A FRACTION OF WALL HEIGHT	0.17	0.21	0.24	0.28	0.34	0.37	0.42	0.43
REDUCED HOOP FORCE DUE TO KEYED WALL (kN) - with fixity of base joint	1.54	2.17	2.58	3.16	4.05	4.57	5.51	5.93
AREA OF STEEL PROVIDED PER 100 mm (mm^2)	9.9	20.18	20.18	30.08	30.08	40.37	50.65	50.65
ACTUAL STRESS ON STEEL PROVIDED (243 MPa ALLOWED for 485 MPa STEEL)	156	108	128	105	135	113	109	117
ACTUAL DESIGN FACTOR OF SAFETY	1.56	2.25	1.9	2.31	1.8	2.15	2.23	2.08

Notes:

1. This table is based on the use of high tensile mesh and wire which has a tensile strength of 485 MPa. If mild steel reinforcing is used then the calculations have to be adjusted accordingly. The tensile strength of mild steel is usually taken as 290 MPa.
2. 1 MPa is the same as 1 N/mm^2
3. In this table the tensile strength of the mortar (typically > 2 MPa) and the tensile strength of the light gauge galvanised mesh which is included in the matrix is not counted.

4.4 Roof

The most economical roof designs are conical or dome shaped, as these shapes limit or entirely exclude bending stresses. Dome shaped roofs require more skill and time to erect than do conically shaped roofs. Modular shuttering systems can be made to support a conical roof, and for this reason this shape is more popular. Some ferrocement designs found in the literature include a central column to support the roof. For the flatter shaped cones this is necessary, but for steeper cones it is not.

The roofs are designed as convex shaped cones, with the angle that the roof makes with the wall being typically approximately 107% (i.e. 17 degrees with the horizontal plane).

In accordance with normal ferrocement design (Refer to SB Watt's book "Ferrocement Water Tanks and their construction", published by Intermediate Technology Publications in 1978, or the American Concrete Institute's "State of the Art Report on Ferrocement", ACI report No. 549, published in 1987), the reservoir roof is a thin shell, with a thickness from as little as 50 mm at the centre to as much as 80 mm at the edge of the roof. Nominally the roof thickness is 60 mm throughout, but for calculating design stresses 75 mm is used. PID has used this design to cover existing unroofed concrete reservoirs, in which case the roof has been simply supported. However, the standard PID practice is to carry one layer of Ref 156 weld mesh through from the walls into the roof to provide additional shear strength at the wall/roof joint.

Being dome shaped the roof is in compression, and therefore the main design consideration is the thrust exerted at the roof/wall joint. To analyse for this thrust, consider a segment of the conical roof 1 metre wide at the roof/wall joint. Assume that the radius of the reservoir is "R", and that the angle made by the roof with the horizontal plane is "A". The length of this roof element is R/cos A.

This weight is supported by the wall, and the thrust component is absorbed by steel reinforcing at the base of the roof, as shown below:

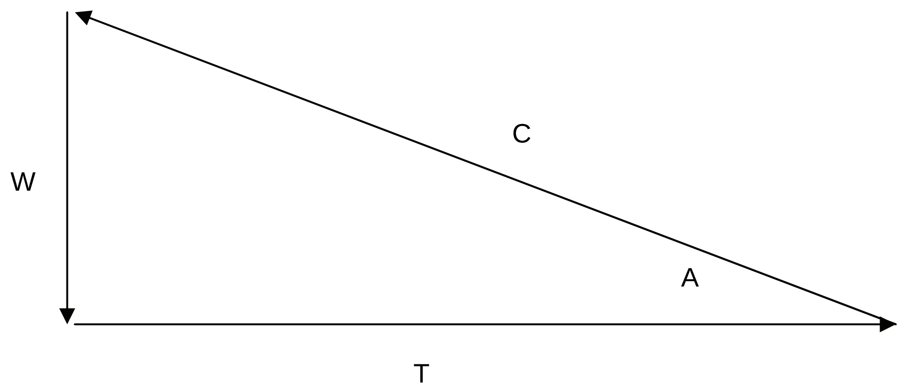

Figure 43: Schematic of forces acting in a segment of a cone shaped roof, with Radius of R metres.

The magnitudes of the forces W, C and T in kN, for an element 0.075 m thick and 1 metre wide at the wall joint, are as follows:

T (Thrust absorbed by hoop steel) = W / tan A

C (Compression in roof element) = W / sin A

W (Weight of roof element) = 0.9375 x R/cosA.

[Note that the roof is not designed to carry a significant live load].

The design splits the roof of the reservoir into two halves, and aggregates the thrust forces for each half together. This thrust is absorbed by the hoop steel on each side of the roof, so the hoop steel must be adequate (after due allowance for safety factors) to absorb half the calculated thrust from half the reservoir roof.

Table 14 below shows the thrust forces which have to be allowed for different reservoir radii, along with the reinforcement that will adequately provide for these forces.

Table 14: Hoop forces in bottom of roof for different tank sizes and provision of reinforcing

Reservoir Radius (m)	Roof Height at Centre (with 17° pitch) (m)	Hoop Force (kN)	Area of high tensile hoop steel required (mm^2)	Area of Ref 156 Mesh included (mm^2)	No. Y12 bars provided	Total Area of Steel Provided (mm2)	Factor of Safety	Shear Stress (MPa)	Compressive Stress (MPa)
2.50	2.6	32	132	50	1	163	2.5	0.031	0.11
3.30	2.9	53	218	50	2	276	2.5	0.041	0.14
4.00	3.1	79	325	50	3	389	2.4	0.049	0.18
4.85	3.3	121	498	50	4	502	2.0	0.059	0.22
5.60	3.6	154	637	50	5	615	1.9	0.069	0.25
6.55	3.9	181	870	50	7	842	1.9	0.080	0.29

Note: Thrust can also be absorbed by the wall, which includes a Y12 bar placed near the top and one or more layers of Ref 156 mesh reinforcing.

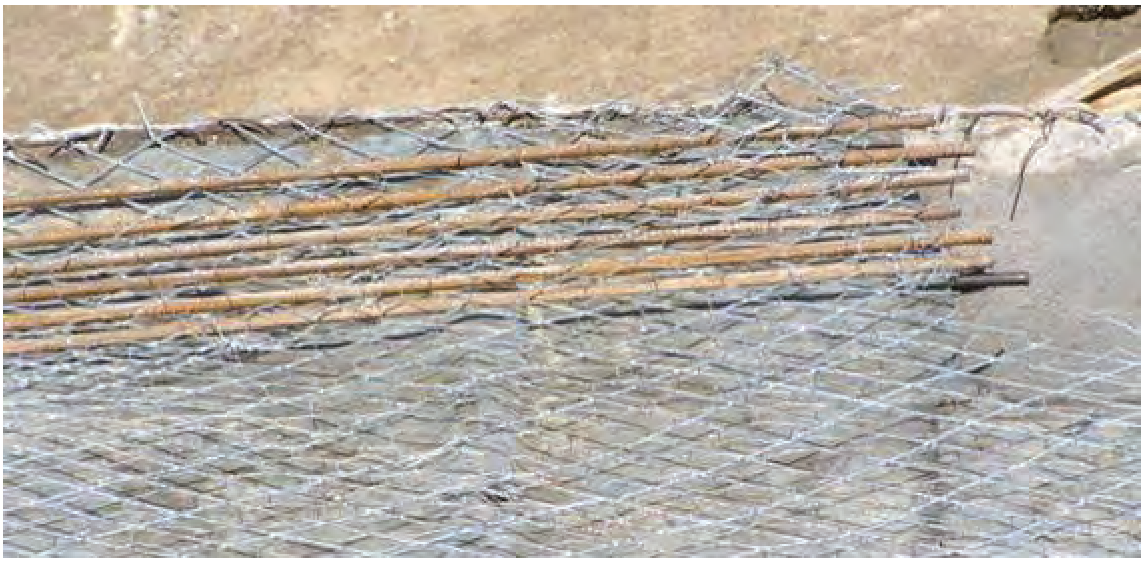

Figure 44: Detail of roof hoop reinforcing provided for a 6 metre diameter reservoir

A further possible mode of failure is **shear** at the roof/wall joint. From the above, the weight of a roof element of thickness 0.075 metres and width one metre at the roof/wall joint is 0.9375 R/cos A in kN. Therefore the shear stress at the wall/roof joint in kN is 0.9375 R /(cos A. x bd), where bd is the cross sectional area subjected to the shear stress. Shear stresses for different roof diameters are shown in Table 14 above. These shear stresses are all less than 0.1 MPa, well below the allowable shear for unreinforced concrete, which is 0.35 MPa for 20 MPa concrete and 0.45 MPa for 30 MPa concrete.

Table 14 also shows the **compression** stresses, which are also very low (especially relative to the compressive strength of the mortar mix, which is typically in excess of 20 MPa).

Over the last eight years this design has been successfully used in numerous reservoirs ranging from 20 kℓ to 220 kℓ in size (with radii ranging from 2 meters to 6.5 meters). Prior to 2002 the design was more domed with a curve rather than an angle at the roof/wall joint. This design did not include any hoop reinforcing in the roof, except for one Y12 bar at the top of the wall and the largest reservoir built had a diameter of 5.6 meters. The design was changed in 2002 to allow for the introduction of a modular roof shuttering system which results in more evenly shaped roofs.

The roof also includes a single layer of 2.5 mm thick 100 x 50 mm galvanised mesh. This is included to prevent cracking but is not structural.

Finite Element Analysis of Thin Shelled Conical/Domed Roof

The method for analysis described above is simplified. In reality a cone or a dome is a complex structure with the different elements interacting with each other in ways that are not expressed in the simple equations above. Structural engineers use finite element analysis to analyse the stresses in complex structures.

A finite element analysis of the forces acting in cone and dome shaped roofs of different diameters and slopes can be found in Annexure 1. The analysis has been repeated for 10 metre and 14 metre diameter roofs for both conical and spherical roofs of slope varying from 12° to 17° to 22°. As is expected, the analyses show that the conical shape is subjected to lower stresses in all respects.

The following worked examples illustrate the relationship between the results of the finite element analyses and the simplified method described above.

i) Hoop stress

According to the results shown on page Annexure 1, page A1, the hoop stress expected at the perimeter of a 10 metre diameter cone shaped roof 0.075 m thick and sloping at 17° is 2 930 kN/m^{2,}, while a metre from the perimeter the stress is 301 kN/m^2. To convert these stresses to a force, average them and multiply by the area which is 1 m by 0.075 m, to get 121 kN. In Table 14 above this is the hoop force given for a 9.7 m diameter reservoir.

ii) Shear stress

According to the results shown on page Annexure 1, page A3, the sheer force expected at the perimeter of a 10 metre diameter cone shaped roof 0.075 m thick and sloping at 17° is 4.73 kN/m. To convert this force to stress, divide by the area over which it acts which is 1 m by 0.075 m, to get 0.063 MPa. In Table 14 above the shear stress given for a 9.7 m diameter reservoir is 0.059 MPa.

iii) Bending stress

According to the results shown on page Annexure 1, page A4, the peak bending moment in a 10 metre diameter cone shaped roof 0.075 m thick and sloping at 17° is 0.553 kNm/m and this occurs at a radius of 4.2 metres. To convert this force to stress, divide by Z where

$$Z = 1/6.\ b.h^2$$

The resulting bending stress is 0.59 MPa.

The finite element analyses appearing in Annexure 1 used a superimposed live load of 0.5 kPa (equivalent to one 50 kg person standing on every square metre of the roof, for example) in addition to the self-weight of the reservoir, which adds approximately 25% to the loading. In practice the only conceivable circumstance which could lead to such a live load is if the roof is used as a vantage point for a soccer match!

A note on the tensile strength of mortar

The tensile strength of concrete or mortar can be derived from the formula[12]

$$\text{Tensile Strength} = 0.5\ (\text{Crushing strength})^{0.67}$$

The crushing strength of the mortar typically used for ferrocement reservoirs is in excess of 20 MPa. Applying the above formula the resulting tensile strength would be 3.72 MPa. This strength is not used in the calculations above for calculating how much hoop reinforcing is required, but it is more than sufficient to ensure that the small bending stresses in the conical shaped roof can be absorbed by the mortar without reinforcement. If the cone is slightly convex in shape these forces will be reduced.

4.5 Mortar mix design

Typically the sand:cement mix for ferrocement varies from 2.5:1 to 3:1 (unlike plaster which is usually 6:1), and the sand is a mix of plaster sand and concrete sand. The resultant matrix is much stronger than conventional plaster or mortar. For example, between February and November 2006 35 cube samples were taken from a set of ferrocement reservoirs built in the Eastern Cape near Matatiele. The median crushing strength of these samples was 30.2 MPa, with a standard deviation of 6.8 MPa, which is comparable to good quality concrete.

It is particularly important that the water:cement ratio is limited to 1:2. The addition of extra water (to make mixing easier for example) weakens the resulting mortar strength.

5. CONSTRUCTION GUIDELINES

Annexure 2 contains PID's *Ferrocement Reservoir Construction Manual*. This was originally drafted in conjunction with Frans Diener of World Vision, an experienced concrete technologist employed by World Vision who for many years built small ferrocement reservoirs for rural communities. The CSIR's first large domed roof ferrocement reservoirs were built with Diener's help, and until 1996 Diener trained PID's reservoir construction teams.

The key points in the manual are as follows:

Foundation preparation:
Before commencement of construction the top soil must be removed, the site must be levelled and all loose material must be removed. The reservoir should not be constructed on uncompacted fill material or on wet material.

Mixing platform and mix:
Using a weak concrete mix a level and clean mixing platform must be constructed to ensure that the materials used in the reservoir are free of soil and organic material. Sand should be screened to remove clay or soil lumps and any organic material. The sand used for the mortar should be neither too coarse nor too fine. Depending on the nature of the locally available sand, some blending of river sand and plaster sand may be required to achieve the right composition. Careful attention must be paid during the mixing process to ensure that the water:cement ratio does not exceed 1:2 and that the mix batches are not larger than can be used in 30 minutes. Under no circumstances must water be added to mortar which has begun to set to make it more workable – such practice weakens the mix.

[12] From Fulton's Concrete Technology, Portland Cement Institute, Sixth Ed. 1986 page 278.

Blinding layer:
A blinding layer of approximately 50 mm thickness is placed on the soil foundation. This gives a clean and firm foundation for setting out the base steel and casting the base.

Pipework:
uPVC should be used for the buried pipework. This is preferred as it will not rust over time, as will ordinary galvanised steel. The outside of the uPVC pipe should be roughened with sandpaper or sand and glue to ensure that it forms a bond with the encasing concrete. The pipe ends should be plugged with paper or plastic to prevent concrete and mortar from falling into the open pipes during the construction of the reservoir. These plugs must be removed when the job is completed.

Concrete base:
Once the base reinforcing steel is in place (lifted 30 to 50 mm above the blinding using plastic or concrete spacers) the base should preferably be cast in one day. This ensures the best protection against seepage through the base.

Wall shuttering and steel:
The method used by PID uses corrugated iron shutters to assist with the wall construction. The use of shutters is, however, not essential. The addition of an extra two layers of fowl netting or fencing mesh to the wall reinforcing stiffens the mesh enough make it possible to plaster into the mesh without a shutter, although a hand held travelling shutter may be used. The advantage of using the shuttering is that it saves on this extra mesh, it speeds the erection of the wall reinforcing and it makes it easier to get an even and regular wall shape. The shutters must be clean and free of any soil or mortar. The reinforcing steel must be well tensioned and each layer must be tied to the layer behind so that there are no loose folds in the mesh.

Outside mortar coats:
The first outside coat should just cover the steel. The second coat should be applied while the first coat is still green, and the first coat must be scoured to ensure a positive bond between the two coats. When these two coats are completed, the reservoir should be kept wet and wrapped in plastic for seven days to aid curing. The second coat need be no more than 10 or 15 mm in thickness. Adding extra layers or thickness to the outside of the reservoir adds nothing to its strength or ability to hold water, and may lead to unsightly cracking or even delamination.

Inside mortar coats:
The inside mortar coats are the key to the water retaining ability of the structure. After the shutters have been stripped the inside surface of the outer layers must be brushed and cleaned of any loose material. The first outside coat should be thick enough to fill the corrugations in the surface with approximately 5 mm of mortar to spare. When dry this coat must be scoured to ensure a strong bond with the second inside coat. The second inside coat should be applied while the first inside coat is still green, i.e. within 24 hours.

Roof:
The roof is the most technically challenging stage in the reservoir construction. It is designed as a thin shell convex cone or dome, with only hoop reinforcing to contain the outward thrust and keep the shell in compression. The key aspects to control are the slope (steeper is stronger – PID specifies a 17° slope at the wall roof joint), the shape (which must be convex, not concave) and the shell thickness (no more than 60 mm is needed and excess thickness may cause local overloading. The roof should ideally be cast in one day.

Sealing:
Although a well made ferrocement reservoir will be water tight, a degree of micro-cracking is normal in thin shell water retaining structures. Typically these cracks soon self-seal. To minimize any seepage through micro-cracks a coat of a sealant such as Brilatex can be applied to the inside of the reservoir.
The wall-base joint may evidence some minor seepage or weeping, which is typically insignificant in terms of flow, but it is unsightly. To minimize the chances of any seepage at this joint a bandage and sealant should be applied to the inside of this joint.

Cleaning up:
The job is not complete until the pipework is complete, with chambers for securing and protecting the valves. The scour outlet should be extended until it daylights and a small brick or stone masonry headwall must be constructed to finish it off. The mixing platform must be broken up and removed, and any further signs of the building operation must be cleaned up and removed. Finally, the pipework must be checked to ensure it is clean and open.

Two coats of good quality acrylic paint applied to the outside of the reservoir will improve its finished appearance.

6. CONCLUSIONS AND RECOMMENDATIONS

Ferrocement differs from conventional reinforced concrete in that

i) no stone aggregate is used in the mix
ii) the steel reinforcing consists of a matrix of small diameter wires and mesh closely spaced, rather than larger diameter bars spaced further apart
iii) minimum thicknesses of ferrocement elements are small, from as little as 10 mm (although 50 to 60 mm is more typical for water retaining structures)
iv) ferrocement is not poured between and moulded by shutters, but is worked by hand into the steel mesh reinforcement

Although perhaps better known for its use in the making of “concrete yachts” and unusually shaped roofs, ferrocement has been used for water retaining structures for several decades. Until the 1990s this usage was limited in South Africa to 5 kℓ reservoirs, although larger ferrocement reservoirs had been built elsewhere in Southern Africa in the 1970s and 1980s. It was engineers working for the CSIR which were the first to try out the use of ferrocement for reservoirs larger than 10 kℓ. In South Africa the first “large” ferrocement reservoirs were built in 1990, being two 40 kl reservoirs at KwaHlophe. These have been out of service for many years due to the decommissioning of the water scheme of which they were a part and are no longer in existence. However in 1992 in an adjoining settlement two 75 kℓ reservoirs were built, and these are still in service and in good condition. Since that time more than one hundred larger (typically 20 to 150 kℓ in size) ferrocement reservoirs have been built in South Africa, principally in KwaZulu-Natal and the Eastern Cape. The most notable are the two 450 kℓ reservoirs built at Osindisweni, which is some 15 km west of Verulam. These latter two are still in service and are in excellent condition.

Comparisons of tender prices show that a saving of 40% is achieved through the use of ferrocement in comparison with conventional reinforced concrete. The question that has concerned engineers, who have to be conservative due to the risks they are required to take with other people’s money, is will these structures last? The inspections that have been conducted during the course of this study have proven that properly designed and built ferrocement reservoirs do not show any signs of deterioration after up to 19 years in service, and therefore there seems no reason to expect them not to last for many more years.

It is normal for ferrocement reservoirs to display a certain amount of microcracking. This is because ferrocement is thin walled and relatively flexible. Mostly these cracks are superficial and limited to the outside of the reservoir, but where they do seep or weep they generally seal through a process of calcification within a period of weeks or months. Calcification and streaking is common with all water retaining structures, up to the largest dams. For those who are not familiar with this material this cracking may be disconcerting. For this reason the application of two coats of a good quality PVA on the outside of the reservoir, refreshed every few years, greatly improves the reservoirs' outward appearance and is recommended.

The design of most of the ferrocement reservoirs built in KwaZulu-Natal, and some of those built in the Eastern Cape, are evolved from one introduced by Graham Simpson, then of the CSIR in Pretoria in 1990, who had himself first used it in the 1970s in the then Rhodesia (now Zimbabwe). Over the years the following design changes have been introduced:

- A domed roof was adopted instead of the catenary shape
- A separate scour outlet was included
- uPVC piping, which does not rust like galvanised steel piping, was used for all pipes cast into the base. [A watertight bond is achieved by scouring the outside pipe wall, or using PVC glue and sand]
- A simple level indicator was included as part of the roof design
- More sophisticated hatches and valve chambers were adopted
- The shape of the roof shuttering (previously shaped by eye and therefore sometimes irregular) was regularized using a support system of telescopic poles and interlocking radial rings. This change straightened the roof slope somewhat so that the new roof design, though still convex, was more cone than dome shaped
- More wire and mesh was added to certain sizes of reservoir to increase the design safety factor and reduce micro-cracking. In particular the importance of vertically oriented steel in the smaller reservoir sizes has been proven and this is now standard for all but the smallest (5 kℓ) reservoirs, where wire and fowl netting suffice
- A bandage and sealant was specified as standard on the wall-floor joint.

Design parameters have been developed for the walls and the roofs, and these have been included with this report. A finite element analysis has been carried out to check the accuracy of the simplified method used for the analysis of the roof stresses, and this has proven the adequacy of the design method used.

Given their significantly lower cost and the evidence of their durability, there is no reason why ferrocement reservoirs cannot be used with confidence. They do not necessarily look as smart or professional as reinforced concrete reservoirs which have the cleaner and straighter off-shutter finish, but particularly in the agricultural and the rural sector where cost is a greater consideration, they have their place. A further advantage in a country with South Africa's high unemployment levels, is that they are more labour-intensive than other types of reservoirs, and they do not require the importation of any expensive materials.

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Annexure 1

A finite element analysis of the stresses in conical and domed ferrocement roofs

ANALYSIS OF DOMED FERROCEMENT RESERVOIRS ROOFS

By Kobus Burger, PrEng
Jeffares & Green (Pty) Ltd **17 March 2009**

Background & terms of reference

Jeffares & Green was appointed by PID to conduct a finite element analysis on domed ferrocement reservoir roofs, refer to their letter dated 2 February 2009. Also as per earlier correspondence (e-mail dated 8 Jan 2009), Jeffares & Green has to furnish PID with results such as: horizontal circumferential stresses, meridian compression stresses, shell moments and shear forces in the dome. As per request two reservoir diameters (diameters of 10 metres and 14 metres) were investigated. The purpose of the investigation was also to determine the variation in stress in domes by varying the centre point heights.

In order to gain an understanding of how the stresses in the roof changes as it's shape moves from a dome with a more pronounced convexity to more shallow dome, three roof scenarios (22 deg, 17 deg & 12 deg offsets with horizontal) for the 10 m & 14 m diameter domes were modelled and investigated. PID's drawing 14631ZS0/303 "Reservoir No3 150 Kl Ferrocement Reservoir" was used as guide to obtain relevant design information such as the roof thickness and shape of the dome.

The mix properties for the walls and the roof as stated on the drawing is a "3:1" mix, as per K. Burger's discussion with Mr D. Still, compression strengths varying between 20-35 MPa were achieved depending on site factors.

As understood, depending on the shuttering system and the height of the dome at the centre point, two shape types may be achieved, namely a spherical form and a conical shaped cone. (Refer to Photographs 1 and 2 below, supplied by PID).

Photograph 1: Spherical Shape

Photograph 2: Conical Shape

Domes

In practice domes are generally spherical and are essentially surfaces of rotation about a vertical axis. A vertical section through this axis in any direction is as a rule an arch of a circle. Another possible form is the conical form, giving a triangular section through its axis of revolution.

The forces maintaining stability of the dome act in two directions. Domes resists vertical forces (gravity & imposed loads) as meridional arch type forces much like a series of pie shaped arches. The dome can be supported solely by vertical reactions if a stiff ring is provided to take the horizontal component of the edge meridional force in the form of hoop forces. The meridians are in compression, and this “outwards” thrust exerted at the roof/wall joint is resisted as tensile forces in the hoop direction.

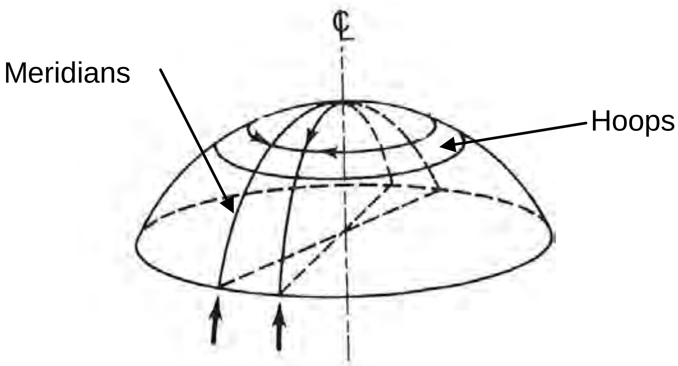


Fig 1: General behaviour & force direction

Modelling

The analysis was done using Prokon’s software and the roofs were modelled using finite elements (shell elements of 75 mm thickness and own weight of 25 kN/m^3)

From the photographs of the reservoirs (and as understood depending on the shuttering system) the shape of the roof may be a smooth curve or having a pointed centre. Thus in each case (for each of the above roof slopes, i.e. 22 deg, 17 deg & 12 deg) a roof either having a smooth spherical shape as well as conical shape were investigated.

The roofs were modelled excluding the effect that the roof and the supporting wall have on one another. In order to obtain the hoop forces near the dome edge, it was firstly assumed that the reservoir roof is simply supported by the wall (vertical support only). This results in horizontal displacement of the dome edge, which then results in a loss of compression stress in meridional direction. This also induced small, but in relation to the thin dome, relative large moments in the meridional direction. It may be argued that once the ring edge is stiffened by hoop reinforcing arresting the hoop forces, these horizontal deflections are also contained, and a second model was analysed having the edge supported in both the vertical & horizontal directions. The results from this second model will give a better representation of the compression stresses and bending moments in the meridional direction.

In addition to its self-weight (1.88 kPa), an imposed load of 0.5 kPa were applied to the entire roof area. The dome behaves linear elastically therefore if the effect of the live load needs to be excluded, the results may be reduced by multiplying it by 0.79.

Table 1 Summary of models analysed:

10 m dia reservoir Conical shape Roof		
Roof slope at support	Support condition	
12°	Vertical only	Vert. & horiz.
17°	Vertical only	Vert. & horiz.
22°	Vertical only	Vert. & horiz.

10 m dia reservoir Spherical shape Roof		
Roof slope at support	Support condition	
12°	Vertical only	Vert. & horiz.
17°	Vertical only	Vert. & horiz.
22°	Vertical only	Vert. & horiz.

14 m dia reservoir Conical shape Roof		
Roof slope at support	Support condition	
12°	Vertical only	Vert. & horiz.
17°	Vertical only	Vert. & horiz.
22°	Vertical only	Vert. & horiz.

14 m dia reservoir Spherical shape Roof		
Roof slope at support	Support condition	
12°	Vertical only	Vert. & horiz.
17°	Vertical only	Vert. & horiz.
22°	Vertical only	Vert. & horiz.

It must be noted the Prokon results does not give an indication on the stability and safety factor against buckling of the dome. Domes are generally stable, nevertheless the domes need to be checked for buckling stability. (Criteria as set by the American Concrete Institute (ACI 334) may be used as a guide.) ACI recommends a minimum safety factor of 4 for domes covering storage areas.

Table 2 Safety Factors against buckling (spherical shaped roof)

As per ACI 334 the following Safety Factors against buckling of the spherical shaped roof having 75 mm thickness were found to be:			
10 m Dia	**SF**	**14 m Dia**	**SF**
12 Degree	96	12 Degree	55
17 Degree	196	17 Degree	103
22 Degree	308	22 Degree	160

Results

The resulting horizontal circumferential (tension/compression) stresses, meridian compression stresses, shell moments & shear forces for each of the sets (see table 1) are given in Appendix 1 of this letter.

In essence the results for the vertical supported domes are applicable regarding the hoop stresses and shear forces but not the meridional stresses and bending moments. The meridional stresses and bending moments results are obtained from the vertical & horizontal supported domes.

The axial system used, is given in Fig.2. The description of the stress/force and axial direction in which they act are given in table 2.

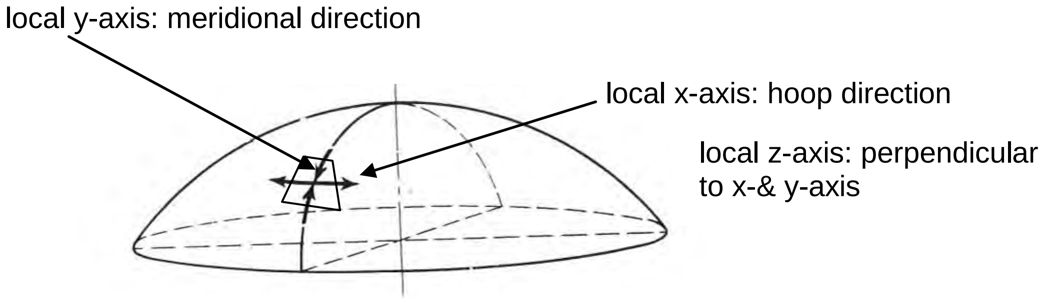


Fig 2: Axial system and forces onto a typical element

Table 3 Description of stresses/forces:

Type	Description	Unit	Model
Shell in plane stress Sx	Hoop -direction	kN/m^2	Vertical supported
Shell in plane stress Sy	Meridional -direction	kN/m^2	Vert. & horiz. supported
Shear forces Vy	Along y- axis direction but perpendicular to it	kN/m	Vertical supported
Bending moments Mx	About x- axis	kNm/m	Vert. & horiz. supported

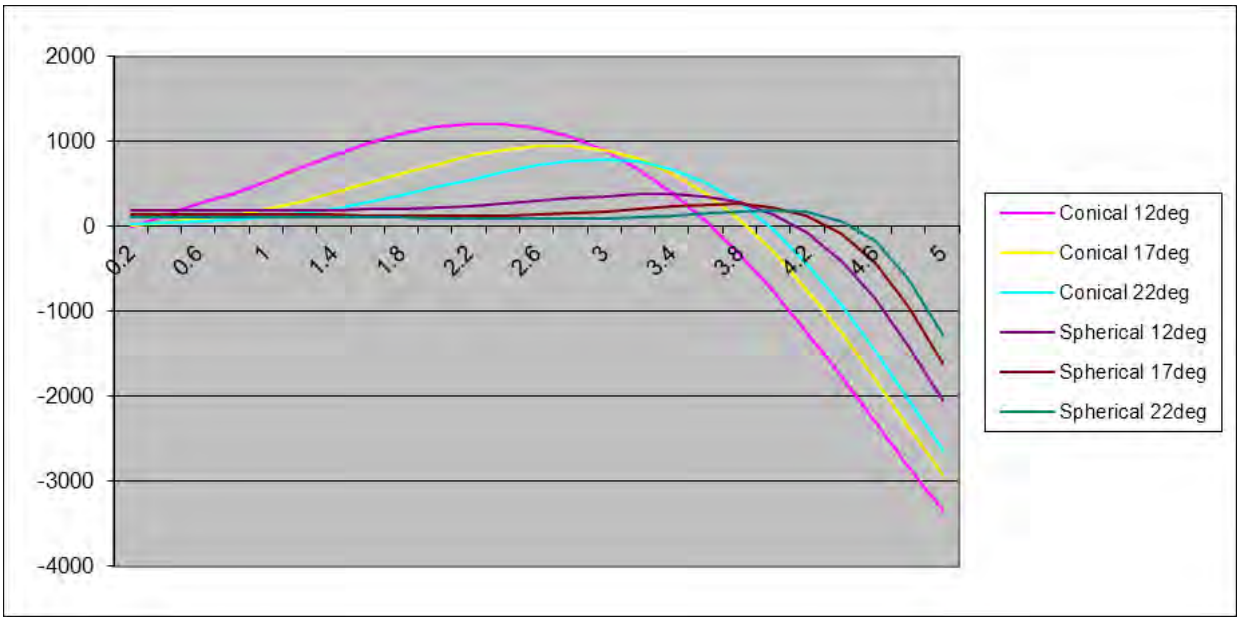
APPENDIX 1

INDEX

A1. 10 m diameter Shell in plane stress Sx

Distance from center point	Conical			Spherical		
	12deg	17deg	22deg	12deg	17deg	22deg
0.2	-4.94	5.06	26.1	194	144	116
0.4	122	37.4	45.2	193	143	115
0.6	269	85.5	65.8	193	142	114
0.8	378	129	83.9	193	140	113
1	529	201	115	193	139	112
1.2	689	291	158	195	136	110
1.4	840	394	214	198	134	108
1.6	975	506	284	203	131	106
1.8	1090	620	366	213	128	103
2	1170	730	457	227	127	99.4
2.2	1210	828	552	246	129	96
2.4	1210	904	643	270	133	92.9
2.6	1160	948	722	298	143	91
2.8	1060	951	776	329	158	91.8
3	895	904	795	358	179	96.8
3.2	676	799	766	379	206	108
3.4	397	629	678	382	235	126
3.6	61.4	390	519	356	259	151
3.8	-328	79.1	283	284	264	178
4	-766	-301	-36.5	148	229	192
4.2	-1250	-747	-438	-73.9	127	170
4.4	-1760	-1250	-915	-400	-78.7	68.3
4.6	-2290	-1790	-1460	-845	-426	-171
4.8	-2820	-2360	-2040	-1410	-939	-614
5	-3340	-2930	-2630	-2060	-1610	-1280

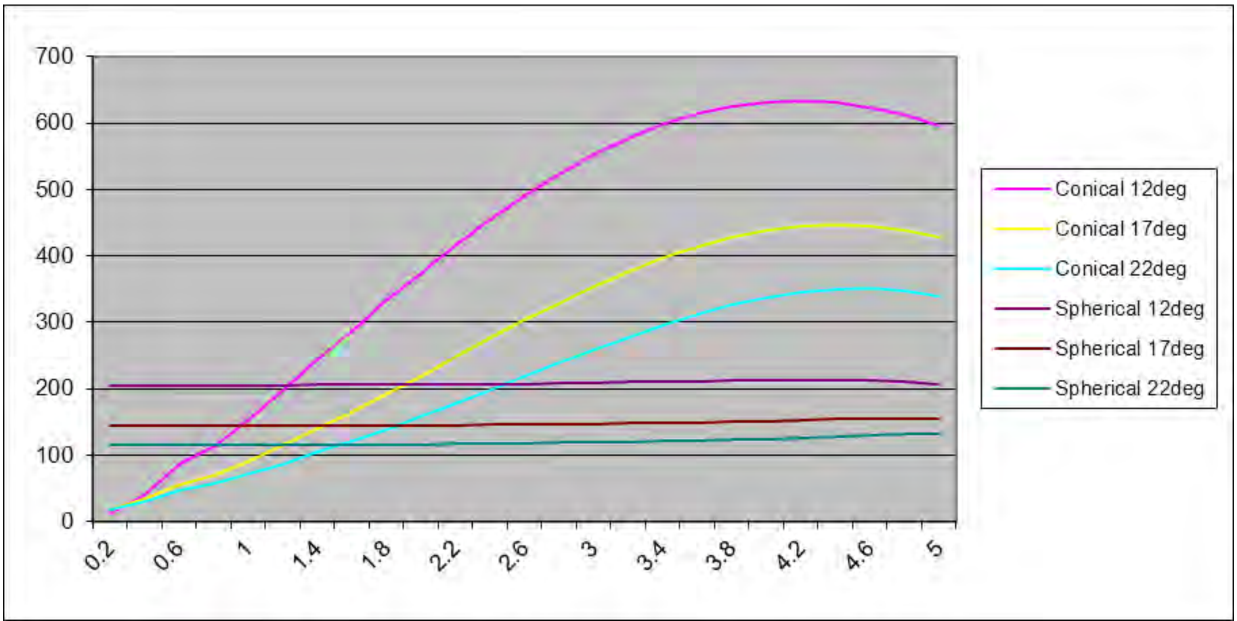
Results are in kN/m^2 (and applicable for self-weight and an imposed load of 0.5 kPa, in the event that only the self-weight stresses are required, the above results are to be multiplied by 0.79.) The results are un-factored working stresses.



A2. 10 m diameter Shell in plane stress Sy

Distance from center point	Conical			Spherical		
	12deg	17deg	22deg	12deg	17deg	22deg
0.2	11.1	18.2	18.3	205	144	116
0.4	41.2	34.1	31.2	205	144	116
0.6	85.9	55.2	46.9	205	144	115
0.8	114	69.9	57.1	205	144	115
1	153	90.4	71.1	205	144	115
1.2	199	115	87.3	205	144	115
1.4	244	140	104	206	144	115
1.6	288	166	121	206	144	116
1.8	332	192	139	206	145	116
2	375	220	158	206	145	116
2.2	416	248	178	206	145	117
2.4	455	276	198	207	146	117
2.6	491	303	219	207	146	118
2.8	523	329	239	208	146	119
3	552	354	259	209	147	119
3.2	577	377	278	210	148	120
3.4	598	397	296	211	148	121
3.6	614	414	313	211	149	122
3.8	625	428	326	212	150	123
4	632	439	338	213	151	124
4.2	634	445	345	213	153	125
4.4	631	447	350	213	154	127
4.6	623	445	351	212	155	129
4.8	612	439	348	210	155	131
5	596	428	339	206	154	131

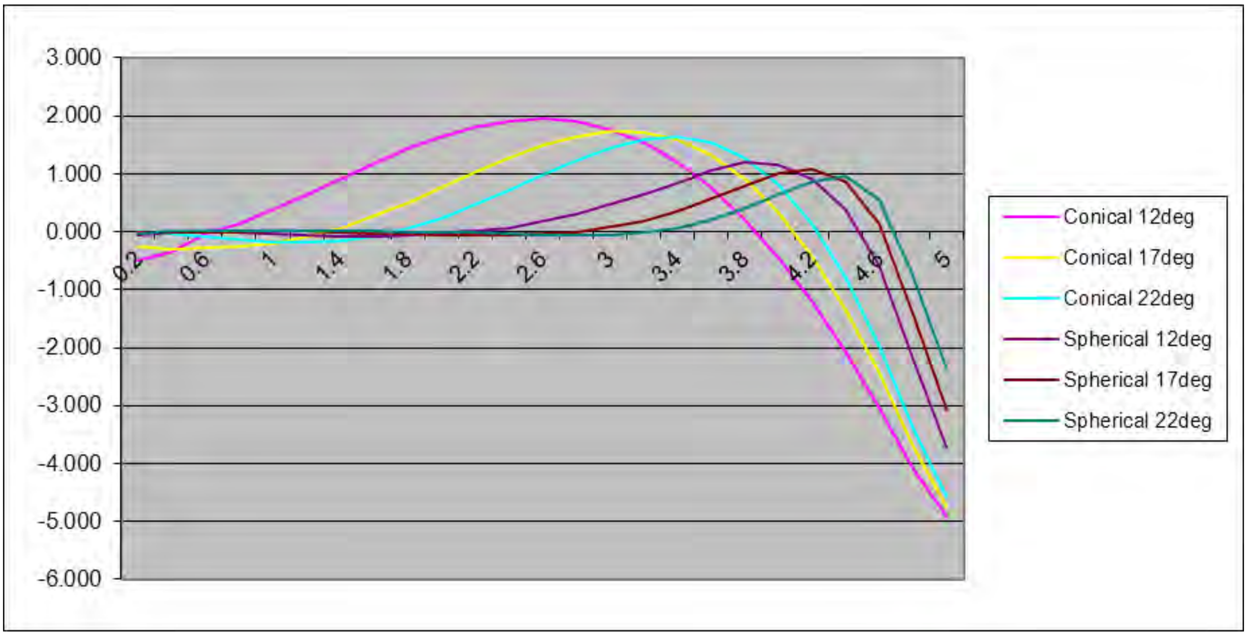
Results are in kN/m^2 (and applicable for self-weight and an imposed load of 0.5 kPa, in the event that only the self-weight stresses are required, the above results are to be multiplied by 0.79.) The results are un-factored working stresses.



A3. 10 m diameter shear forces Vy

Distance from center point	Conical			Spherical		
	12deg	17deg	22deg	12deg	17deg	22deg
0.2	-0.506	-0.253	0.010	-0.055	-0.017	-0.024
0.4	-0.355	-0.310	-0.033	-0.015	0.001	0.011
0.6	-0.045	-0.281	-0.092	-0.015	-0.003	0.006
0.8	0.131	-0.263	-0.135	-0.018	0.001	0.003
1	0.369	-0.207	-0.178	-0.034	0.010	0.010
1.2	0.641	-0.095	-0.192	-0.059	0.007	0.007
1.4	0.917	0.065	-0.159	-0.072	-0.015	0.003
1.6	1.188	0.269	-0.076	-0.079	-0.041	0.005
1.8	1.435	0.505	0.058	-0.061	-0.044	-0.009
2	1.647	0.761	0.239	-0.014	-0.051	-0.020
2.2	1.810	1.022	0.463	0.014	-0.059	-0.019
2.4	1.915	1.269	0.715	0.066	-0.050	-0.037
2.6	1.950	1.485	0.979	0.183	-0.042	-0.051
2.8	1.904	1.646	1.232	0.310	-0.003	-0.048
3	1.769	1.733	1.446	0.467	0.078	-0.050
3.2	1.538	1.723	1.590	0.647	0.183	-0.017
3.4	1.204	1.596	1.633	0.834	0.356	0.072
3.6	0.764	1.333	1.539	1.046	0.579	0.204
3.8	0.214	0.918	1.277	1.191	0.802	0.394
4	-0.444	0.338	0.816	1.160	0.997	0.634
4.2	-1.209	-0.415	0.132	0.913	1.077	0.874
4.4	-2.076	-1.343	-0.794	0.379	0.868	0.948
4.6	-3.037	-2.444	-1.967	-0.597	0.129	0.552
4.8	-4.082	-3.707	-3.384	-2.185	-1.411	-0.755
5	-4.905	-4.730	-4.565	-3.727	-3.060	-2.374

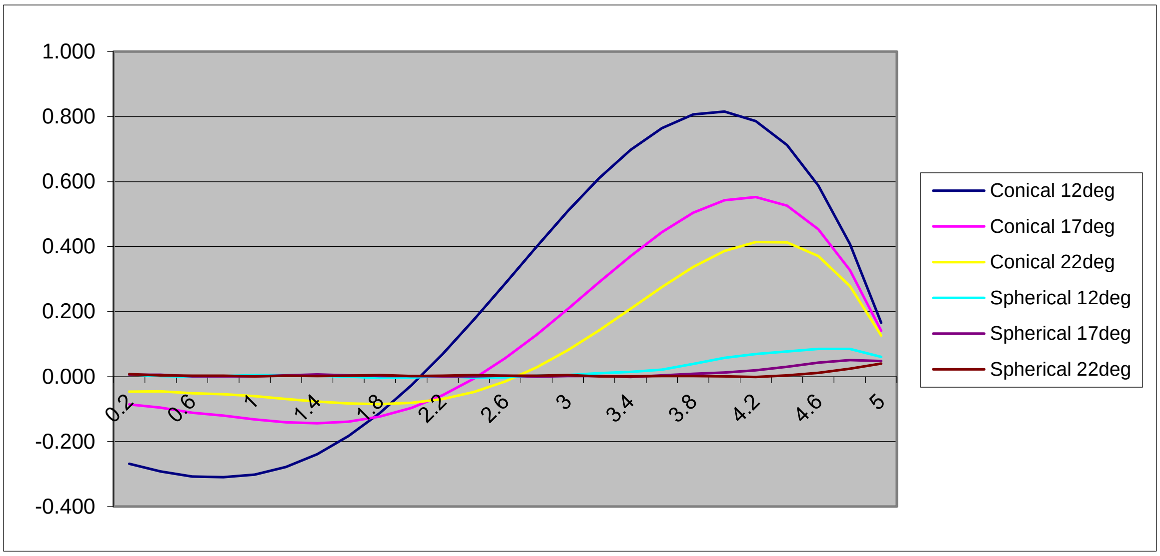
Results are in kN/m^2 (and applicable for self-weight and an imposed load of 0.5 kPa, in the event that only the self-weight stresses are required, the above results are to be multiplied by 0.79.) The results are un-factored working stresses.



A4. 10 m diameter Bending moments Mx

Distance from center point	Conical			Spherical		
	12deg	17deg	22deg	12deg	17deg	22deg
0.2	-0.268	-0.085	-0.046	0.007	0.005	0.007
0.4	-0.292	-0.095	-0.045	0.002	0.005	0.004
0.6	-0.308	-0.112	-0.052	0.000	0.001	0.003
0.8	-0.309	-0.120	-0.054	0.002	0.000	0.002
1	-0.301	-0.131	-0.060	0.005	0.001	0.001
1.2	-0.278	-0.141	-0.069	0.004	0.004	0.002
1.4	-0.239	-0.144	-0.077	0.003	0.007	0.002
1.6	-0.183	-0.139	-0.083	0.000	0.004	0.003
1.8	-0.112	-0.123	-0.085	-0.004	0.001	0.004
2	-0.027	-0.096	-0.081	-0.004	0.002	0.002
2.2	0.070	-0.057	-0.069	0.002	0.001	0.002
2.4	0.175	-0.006	-0.047	-0.003	0.001	0.004
2.6	0.287	0.056	-0.015	-0.002	0.003	0.002
2.8	0.399	0.129	0.028	0.003	0.000	0.003
3	0.509	0.208	0.081	0.005	0.001	0.004
3.2	0.610	0.290	0.142	0.011	0.001	0.000
3.4	0.697	0.371	0.209	0.014	-0.001	0.001
3.6	0.765	0.444	0.276	0.021	0.004	0.002
3.8	0.806	0.504	0.337	0.039	0.008	0.002
4	0.816	0.543	0.386	0.058	0.012	0.000
4.2	0.786	0.553	0.414	0.069	0.019	-0.001
4.4	0.713	0.526	0.413	0.077	0.031	0.004
4.6	0.588	0.454	0.371	0.085	0.043	0.012
4.8	0.408	0.328	0.279	0.085	0.051	0.024
5	0.166	0.141	0.126	0.061	0.048	0.040

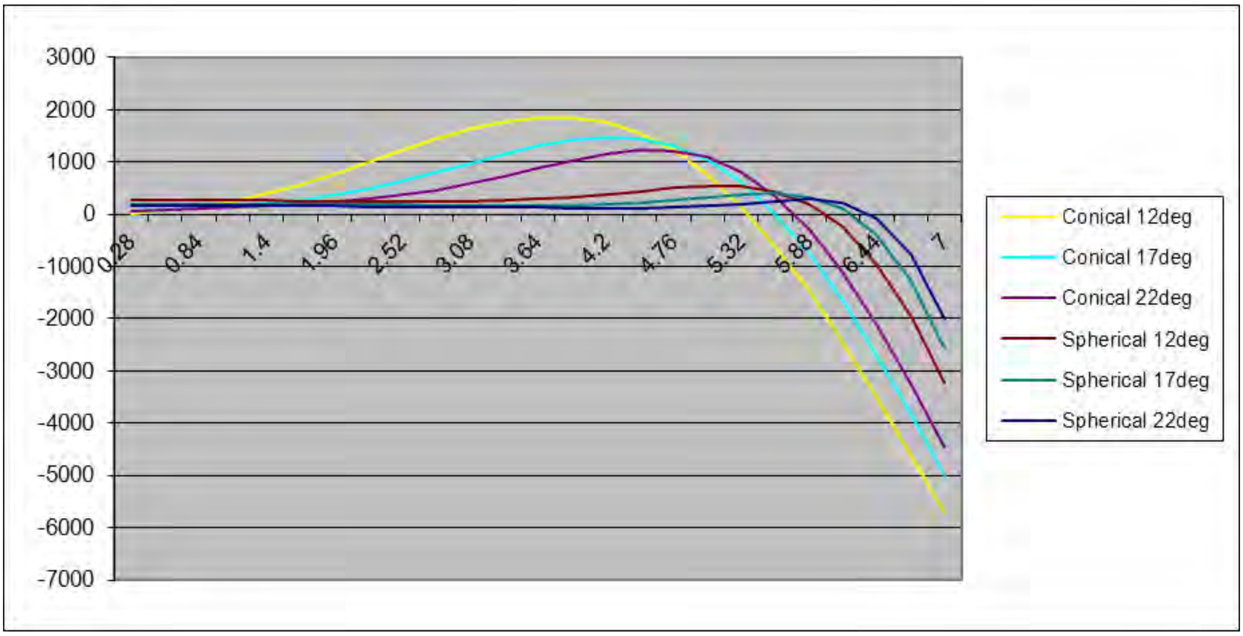
Results are in kNm/m (and applicable for self-weight and an imposed load of 0.5 kPa, in the event that only the self-weight stresses are required, the above results are to be multiplied by 0.79.) The results are un-factored working stresses.



A5. 14 m diameter Shell in plane stress Sx

Distance from center point	Conical			Spherical		
	12deg	17deg	22deg	12deg	17deg	22deg
0.28	8.5	52.5	44.9	273	201	161
0.56	76.8	92.1	87.8	271	200	161
0.84	166	127	121	268	199	159
1.12	253	157	142	265	197	157
1.4	394	206	170	260	195	155
1.68	572	278	203	255	192	154
1.96	776	372	244	251	189	152
2.24	996	489	298	247	185	150
2.52	1220	632	373	245	181	146
2.8	1440	798	471	247	176	143
3.08	1630	977	594	254	171	138
3.36	1770	1150	735	269	167	133
3.64	1860	1310	887	294	167	126
3.92	1860	1430	1030	332	172	120
4.2	1770	1480	1160	382	187	114
4.48	1560	1460	1230	442	215	115
4.76	1220	1310	1210	502	258	127
5.04	753	1040	1090	543	314	155
5.32	145	611	811	537	370	201
5.6	-599	16.4	364	442	395	255
5.88	-1470	-744	-278	204	334	288
6.16	-2450	-1660	-1110	-241	104	227
6.44	-3510	-2700	-2110	-953	-401	-65.4
6.72	-4610	-3830	-3250	-1960	-1280	-770
7	-5720	-5000	-4460	-3230	-2550	-2010

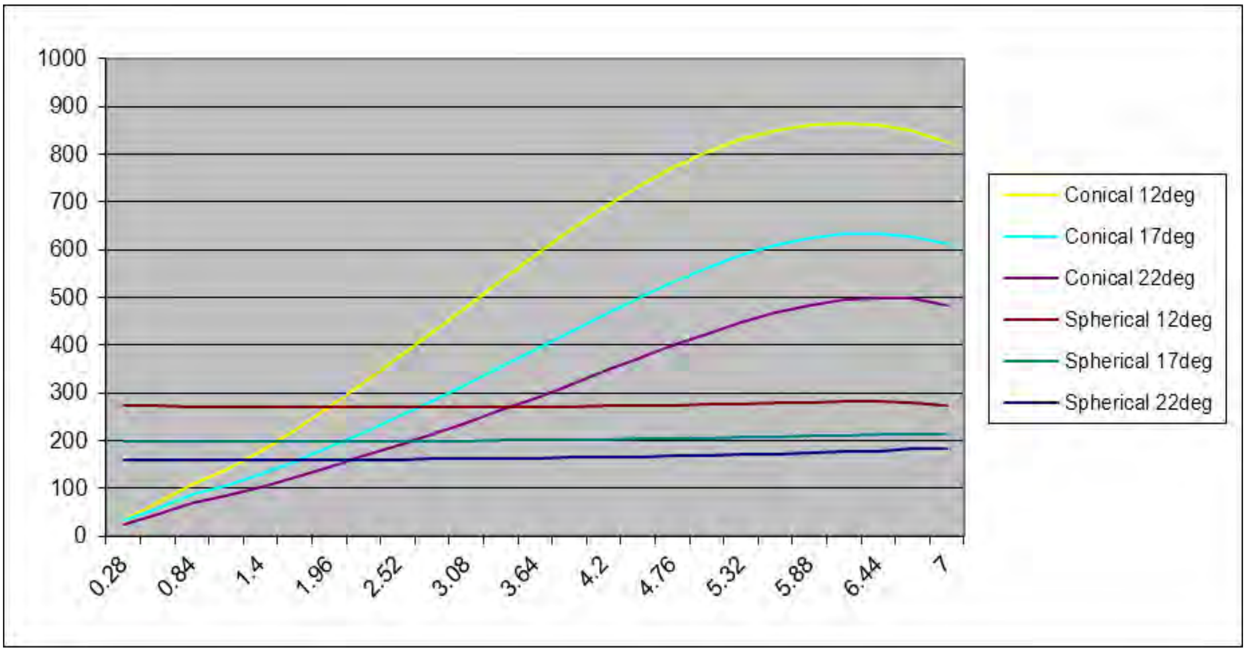
Results are in kN/m^2 (and applicable for self-weight and an imposed load of 0.5 kPa, in the event that only the self-weight stresses are required, the above results are to be multiplied by 0.79.) The results are un-factored working stresses.



A6. 14 m diameter Shell in plane stress Sy

Distance from center point	Conical			Spherical		
	12deg	17deg	22deg	12deg	17deg	22deg
0.28	34.5	32.8	24.8	273	200	161
0.56	69.3	58.9	46.5	273	200	161
0.84	109	86.7	69.3	272	200	161
1.12	138	105	83.6	272	200	161
1.4	179	131	104	272	199	161
1.68	223	158	125	271	199	161
1.96	271	188	146	271	199	161
2.24	322	218	168	271	199	161
2.52	374	250	191	271	199	161
2.8	428	283	214	271	200	162
3.08	482	318	238	271	200	162
3.36	536	354	264	271	201	163
3.64	589	391	290	272	201	164
3.92	640	428	317	272	202	165
4.2	688	464	345	273	203	166
4.48	732	499	372	274	204	167
4.76	771	533	400	275	204	168
5.04	804	563	425	277	206	169
5.32	831	589	449	278	207	171
5.6	851	610	469	279	208	173
5.88	862	625	485	281	210	174
6.16	866	633	495	282	212	177
6.44	862	635	500	282	214	179
6.72	850	629	498	280	215	183
7	827	612	485	275	213	183

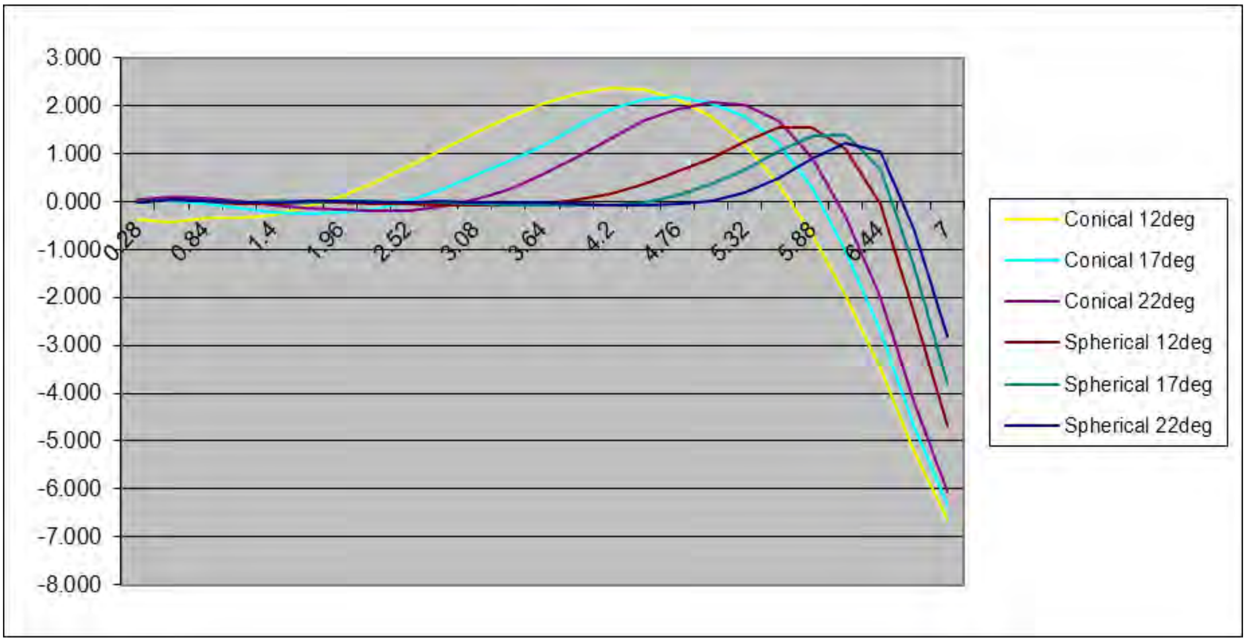
Results are in kN/m^2 (and applicable for self-weight and an imposed load of 0.5 kPa, in the event that only the self-weight stresses are required, the above results are to be multiplied by 0.79.) The results are un-factored working stresses.



A7. 14 m diameter Shear forces Vy

Distance from center point	Conical			Spherical		
	12deg	17deg	22deg	12deg	17deg	22deg
0.28	-0.367	0.047	0.051	-0.008	-0.005	-0.017
0.56	-0.434	0.028	0.110	0.051	0.036	0.033
0.84	-0.347	-0.053	0.078	0.016	0.015	0.011
1.12	-0.331	-0.144	0.008	-0.031	-0.002	-0.007
1.4	-0.279	-0.227	-0.064	-0.035	0.000	0.000
1.68	-0.118	-0.247	-0.122	-0.007	-0.006	0.007
1.96	0.115	-0.230	-0.173	-0.026	0.001	0.008
2.24	0.395	-0.158	-0.202	-0.056	0.006	-0.003
2.52	0.719	0.009	-0.186	-0.054	-0.008	-0.001
2.8	1.069	0.245	-0.095	-0.070	-0.018	0.003
3.08	1.424	0.533	0.050	-0.080	-0.036	-0.005
3.36	1.759	0.840	0.264	-0.077	-0.061	-0.006
3.64	2.047	1.176	0.561	-0.049	-0.066	-0.021
3.92	2.259	1.569	0.915	0.032	-0.067	-0.053
4.2	2.367	1.919	1.325	0.163	-0.063	-0.070
4.48	2.341	2.150	1.683	0.373	-0.012	-0.068
4.76	2.152	2.200	1.944	0.629	0.122	-0.044
5.04	1.777	2.066	2.092	0.915	0.359	0.030
5.32	1.192	1.794	2.018	1.265	0.678	0.198
5.6	0.381	1.236	1.683	1.548	1.045	0.493
5.88	-0.666	0.302	0.935	1.557	1.367	0.898
6.16	-1.952	-1.016	-0.299	1.109	1.393	1.238
6.44	-3.473	-2.698	-1.979	-0.055	0.698	1.038
6.72	-5.214	-4.680	-4.176	-2.308	-1.351	-0.564
7	-6.623	-6.335	-6.074	-4.666	-3.788	-2.816

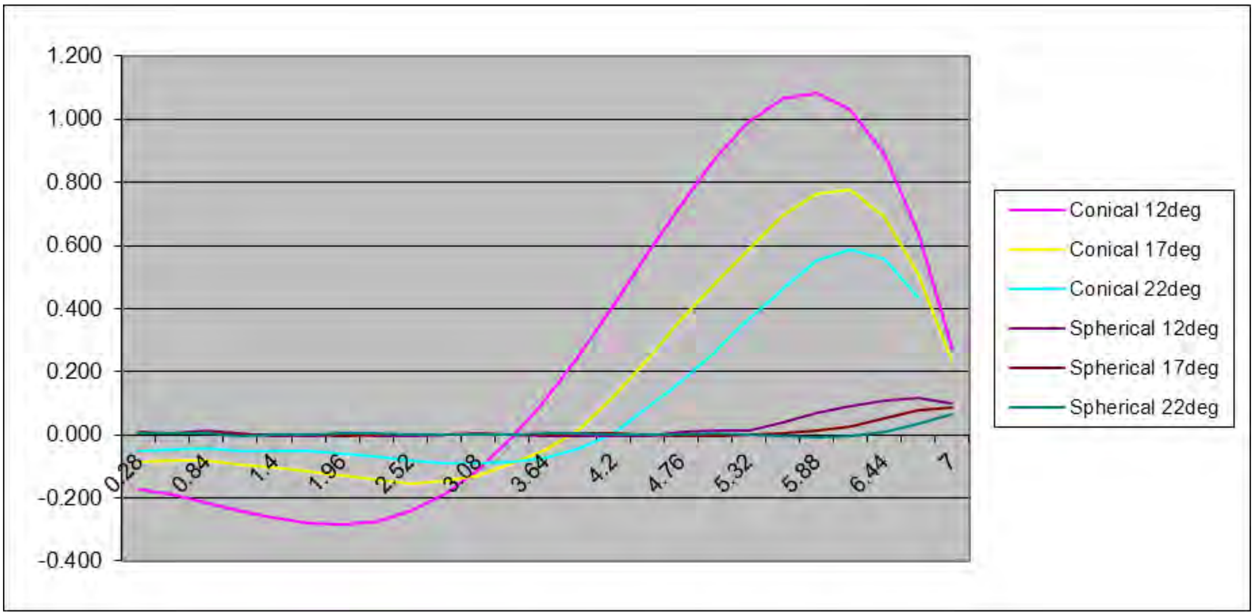
Results are in kN/m (and applicable for self-weight and an imposed load of 0.5 kPa, in the event that only the self-weight stresses are required, the above results are to be multiplied by 0.79.) The results are un-factored working forces.



A8. 14 m diameter Bending moments Mx

Distance from center point	Conical			Spherical		
	12deg	17deg	22deg	12deg	17deg	22deg
0.28	-0.172	-0.084	-0.051	0.007	0.005	0.006
0.56	-0.190	-0.079	-0.047	0.006	0.007	0.006
0.84	-0.217	-0.083	-0.043	0.012	0.005	0.004
1.12	-0.242	-0.093	-0.049	0.006	-0.001	-0.003
1.4	-0.264	-0.104	-0.050	-0.004	0.002	-0.001
1.68	-0.281	-0.115	-0.051	-0.003	0.002	0.001
1.96	-0.285	-0.126	-0.058	0.005	-0.002	0.004
2.24	-0.273	-0.144	-0.068	-0.002	0.001	0.004
2.52	-0.242	-0.154	-0.080	-0.002	0.003	0.001
2.8	-0.188	-0.148	-0.090	0.002	0.002	0.003
3.08	-0.111	-0.127	-0.088	0.001	0.004	0.002
3.36	-0.010	-0.089	-0.086	0.003	0.002	0.002
3.64	0.113	-0.046	-0.071	-0.002	0.000	0.006
3.92	0.255	0.020	-0.042	-0.004	0.005	0.005
4.2	0.411	0.120	0.007	-0.004	0.006	0.001
4.48	0.572	0.238	0.086	-0.003	0.001	0.000
4.76	0.730	0.367	0.169	0.009	-0.004	0.002
5.04	0.874	0.480	0.266	0.012	-0.005	0.004
5.32	0.991	0.586	0.367	0.016	0.001	0.003
5.6	1.066	0.693	0.464	0.039	0.006	-0.002
5.88	1.084	0.763	0.555	0.068	0.013	-0.006
6.16	1.031	0.776	0.587	0.090	0.028	-0.003
6.44	0.888	0.690	0.556	0.107	0.051	0.012
6.72	0.642	0.509	0.438	0.116	0.079	0.033
7	0.276	0.235	0.207	0.099	0.087	0.067

Results are in kNm/m (and applicable for self-weight and an imposed load of 0.5 kPa, in the event that only the self-weight stresses are required, the above results are to be multiplied by 0.79.) The results are un-factored working stresses.



Annexure 2

PID Ferrocement Reservoir Construction Manual

FERROCEMENT RESERVOIR CONSTRUCTION MANUAL

written by

F. Diener, D. Still, A. Broughton, and A. Butler

June 2011

PARTNERS IN DEVELOPMENT
PID
CONSULTING ENGINEERS
PROJECT MANAGERS
(Pty) Limited

WATER
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Foreword

I first encountered the concept of ferrocement reservoir construction in 1990 when on secondment from the Department of Water Affairs to the CSIR's newly formed Appropriate Technology Group. One of the group's engineers, Pavel Sadzik, was busy making a set of education videos on appropriate technology topics, including how to protect a spring, how to build a household slow sand filter, and how to construct a 5 kℓ ferrocement reservoir. The expert ferrocement technologist who he had consulted to assist him in this task was Frans Diener of World Vision, an NGO which had been engaged for many years in spring protection and small reservoir construction in South Africa's rural areas. Frans was a concrete technologist raised and educated in Germany, and had established rigorous standards for his reservoirs. When in 1992 I became responsible for the construction of the first large ferrocement reservoirs in South Africa, I turned to Frans to assist with the training and supervision of the work crew, which was entirely recruited from the rural community of KwaNyuswa, Ndwedwe District, where we were building a new water supply system. Frans and I continued to work together after I founded *Partners in Development* (PID) in 1993, and he trained our reservoir construction teams until 1996.

Over the years as we have learned from experience our methodology has evolved somewhat. The changes have included:

i) The addition of a mandatory concrete blinding layer beneath the reservoir floor
ii) The inclusion of a dedicated scour outlet
iii) The change from six layers of mortar in the walls to four, which is more in line with the thin reinforced shell design philosophy of ferrocement and which limits the occurrence of delamination of the outer layer
iv) The use of a sealant and bandage to reduce the incidence of weeping or seeping from the base-wall joint
v) The switch from a dome shaped roof to an only slightly convex cone shaped roof, to improve the regularity of the roof shape and to speed the erection of the roof shuttering.

Since 1996 many PID engineers, technicians and foremen have looked after our ferrocement reservoir construction work. In particular I would like to acknowledge the contributions made by Jenny Appleton, Andrew Broughton, Andrew Butler, David Gadd, Gordon Garcia, Malcolm Hopkins, Bongane Mabaso, Barend Marx, Muzi Mthetwa, Stephen Nash, Raishan Naidu, Peace Ndlovu, and Brian Ngcobo.

The original design which I used in 1992 was adopted from one which the CSIR's Graham Simpson had previously developed in Zimbabwe. Graham later joined VKE (now VelaVKE) and has been responsible for the construction of large ferrocement reservoirs on rural water projects. His construction method, which makes use of more steel but dispenses with shuttering, is described in his 1998 Concrete Society (KZN) paper *Introduction to ferrocement reservoir design and construction.*

Many engineers in South Africa are still largely ignorant of ferrocement reservoir technology. The CSIR's contribution in pioneering the original *large* ferrocement reservoirs in South Africa in the early 90's should therefore be noted. The Water Research Commission of South Africa must also be thanked for affording PID the opportunity to assess the long term performance of these reservoirs in the field (the oldest of which are now 19 years old and which show no sign of deterioration, as discussed in the report *Ferrocement Reservoirs, a South African Perspective*).

David Still, PrEng
Partners in Development (Pty) Ltd
June 2011

1. Clearing the Site and Excavating the Base

1.1 Mark out the area to be cleared for the reservoir by scribing a circle at least 500 mm larger than the radius of the base (See Table 1 of Appendix A for the base radii of various sizes of ferrocement reservoir). If the reservoir is being constructed on a slope additional working space may be required in which case the excavation radius should be increased enough for wheelbarrow access all around the perimeter.

1.2 After clearing all vegetation, remove the topsoil and stockpile. Excavate down to solid foundation material. If the base is to be cast on any fill material (to be avoided if at all possible) it should be compacted in 100 mm layers using stampers or whackers with sufficient moisture added to the fill material until it is quite firm. If the foundation material is of a type that is prone to cracking, swelling or heaving (i.e. certain clays), specialist advice should be sought.

1.3 A simple method of achieving the correct level is to place pegs at 1 m grid spacings with their tops 300 mm above the final level. As the blade of a spade is 300 mm wide, workers can use this as a gauge.

1.4 To avoid breaking up or rutting the surface, wheelbarrows should travel on planks within the excavated area.

1.5 Excavate trenches for the outlet, overflow and scour pipework. Figure 1 below shows the excavation after final levelling for a 120 kℓ ferrocement reservoir with trenches for the scour and overflow pipework visible in the foreground.

Figure 1. The excavation after final levelling for a 120 kℓ reservoir

1.6 uPVC pipes, which do not corrode over time like steel pipes, are recommended for the pipework to be cast into the reservoir base. To help create a bond between the concrete and uPVC pipe, the pipe should be coated with PVC glue and covered with river sand while the glue is still tacky. Alternately the pipes' outer surface can be roughened using a coarse grade of sandpaper.

1.7 In order to reduce the possibility of leakage, it is desirable to encase any pipework below the base in concrete. Figure 2 below shows the scour and overflow pipework protruding from the base of a reservoir. Note the concrete encasement around both pipes.

Figure 2. Scour and overflow pipes showing concrete encasement around pipes

2. The Mixing Platform

2.1 While excavation is taking place, the mixing platform can be cleared and levelled. The platform will ensure that the concrete and mortar that is used in the reservoir is free of extraneous soil and organic matter which if present could result in structural weakness or leaks.

2.2 Cast a 5000 mm x 3000 mm x 50 mm mortar slab over the mixing platform (make a 1:4 mortar mix using coarse river sand).

3. Hand Mixing of Concrete and Plaster

3.1 Wet the mixing platform thoroughly without allowing pools of water to form.

3.2 Spread the sand evenly in the form of a sausage down the platform. Now spread the cement evenly over the sand. Mix back and forth until the colour is uniform.

3.3 Move the mixed materials away from the centre to form a pond. Add half of the required water[1]. Mix the water with the dry mix (3.2) and then slowly add the remaining water. Take care not to add too much water to the mixture. Figure 3 shows an example of this technique being employed.

[1]The total water should be two and a half 10 litre buckets per 50 kg bag of cement.

Figure 3. When mixing concrete or plaster, take special care to mix the sand and cement evenly before the addition of water, add water incrementally and don't use too much water

3.4 If concrete is required, stone should now be added to the mixture. Note that no more water should be added to the mixture. Generally, labour will be opposed to this method of mixing concrete. However, it has been found that following this method helps to prevent excessive water being added to the concrete during hand mixing.

3.5 Wet mixing should be continued until the colour and consistency of the mix is uniform throughout.

3.6 When preparing concrete or plaster, do not add water to a dry mix unless it can be used up within half an hour.

3.7 Under no circumstances should concrete or plaster that has begun to harden be used. Adding water to an old mix should never be employed as this seriously weakens the mix. The best way to avoid this happening is to prepare the mixes in small batches.

4. Casting the Blinding

4.1 Once the base has been excavated a 50 mm concrete blinding is cast. The blinding ensures that the base proper is cast on a clean and strong surface.

4.2 Wet the ground before placing the blinding.

4.3 Shutter boxes must be placed around all pipework passing through the base to avoid encasing them in lower grade blinding concrete. Plug all pipework to stop concrete and other dirt from entering and clogging the pipes (and do not forget to remove these plugs when the job is complete!)

4.4 Use a 1:5:5 concrete mixture (1 bag of cement to 2.5 wheelbarrows of stone to 2.5 wheelbarrows of sand).

4.5 Start by placing concrete at the furthest point from the mixing platform. Figure 4 below shows the early stages of casting the blinding for a 120 kℓ reservoir.

Figure 4. Casting of the blinding for a 120 kℓ reservoir

4.6 To achieve the correct level, hammer short pegs into the ground so that the top of the pegs are at the desired level. The pegs must be removed once the blinding has been cast.

4.7 To avoid disturbing the foundation, the wheelbarrows should travel on planks.

4.8 Extend the blinding beyond the radius of the base to allow working space (500 mm to 1000 mm). The completed blinding for a 120 kℓ reservoir is shown in Figure 5. This blinding was considerably thicker than normal due to the reservoir being constructed on an uneven rock surface (a normal blinding would only be 50 mm thick). Note also the open sections left in the concrete for the outlet pipe (on left) and scour and overflow pipes (on right).

Figure 5. Example of a completed blinding showing the open section left for the pipework (these will only be closed during the casting of the base). Note that in this instance the blinding was much thicker than normal on the one side due to the rock founding material being uneven.

5. Fixing the Base Reinforcement

5.1 Sweep clean the blinding surface and mark (scratch) the circumference of the base on the blinding. Table 1 in Appendix A provides the base radii for various sizes of ferrocement reservoir.

5.2 Mark (scratch) the circumference of the ring reinforcing on the blinding (only used for reservoirs > 15 kℓ. The ring reinforcing has a radius of 30 mm greater than the mould radius (See Table 1 in Appendix A for mould radii used for various sizes of ferrocement reservoir).

5.3 Divide the circumference of the ring reinforcing into α equal parts to position the wall starter bars where α is determined by the size of the reservoir. See Table 2 in Appendix A for values of α for various sizes of ferrocement reservoir.

5.4 Position the Ref 193 weld mesh sheets as per the plan (ref 193 is 200 mm square mesh with 5.6 mm diameter high tensile bars). The pieces of weld mesh should have a minimum overlap of 200 mm. Note, for reservoirs up to 10 kℓ in size it is not necessary to include Ref 193 mesh in the base.

5.5 Fix the Y12 wall starter bars to the mesh at the positions marked in 5.3 above. Note that reservoirs from 5 to 15 kℓ in size do not require wall starter bars.

5.6 Fix the Y12 base ring reinforcing to the corners of the wall starter bars. Note that reservoirs from 5 to 15 kℓ in size do not require the base ring reinforcing.

5.7 Place a grid of 30 mm high concrete or plastic spacers under the mesh to give a minimum cover of 20 mm under the mesh.

5.8 Tie a 40 mm HDPe pipe onto the outside of the starter bars to form a key on removal. The HDPe pipe should be positioned so that its <u>centre</u> line will be flush with the final level of the base. Figure 6 below shows the positioning of the pipe key relative to the wall starter bars and ring reinforcing.

Figure 6. Positioning of the pipe key relative to the wall

5.9 Place short planks or concrete building blocks as shuttering around the circumference of the base. Steel pegs must be used to pin these shutters in place while the base is cast.

6. Casting the Concrete Base

6.1 Prepare a concrete mix of 1:3:3 (2 bags of cement to 3 wheelbarrows of river sand to 3 wheelbarrows of 13 mm stone).

6.2 Wet the blinding thoroughly without creating puddles before placing the concrete.

6.3 Place scaffold planks on top of the reinforcing for the wheelbarrows to travel on.

6.4 Start casting on the far side of the base and work towards the mixing area. Figure 7 shows the casting of the base of a 50 kℓ reservoir. Note the portion of base that has already been cast relative to the position of the mixing area.

Figure 7. Casting of the base of a 50 kℓ reservoir

6.5 Compact and float the concrete with wooden floats. Use floating aids to achieve the correct base depth (100 mm). If no floating aids are available, they can be constructed using a 50 x 50 x 2100 mm plank. Cut three pieces (50 mm in length) and nail two of them to the end of the other piece as supporting feet in order to provide a 100 mm gauge for the base depth.

6.6 The base must be cast in one day. If this is not done the base could leak at the interface between the concrete cast on different days. For larger reservoirs it is advisable to hire a concrete mixer or additional labour to make this possible.

6.7 The concrete should be cured for at least 7 days. The only practical way of doing this is to wet the base at least three times daily. Figure 8 shows the completed base of the same reservoir shown in Figure 7 above.

Figure 8. Completed base of a 50 kℓ ferrocement reservoir

7. Erecting the Wall Shuttering

7.1 Remove the HDPe pipe. Clean the concrete key and remove all loose material.

7.2 Divide the base into β equal sectors and mark these on the edge of the base (so that they are visible after the wall has been constructed). β is the number of 1.5 metre long mould panels that will be required in order to cast the walls of the reservoir and will vary depending on the size of the reservoir being built. See Table 2 in Appendix A for values of β for various sizes of ferrocement reservoir.

7.3 Clean and lightly oil the mould panels with shutter oil (if no shutter oil is available old engine oil mixed with diesel will suffice).

7.4 Erect two mould panels against the inside of the key and using binding wire tie a 75 x 100 x 2000 mm piece of timber between the angle irons of each panel where they overlap. Note that there are two types of panels, those with the angle irons bolted to the panel edge, and those with the angle irons offset. In order for these panels to overlap they should be alternated as one works around the circumference of the reservoir.

7.5 Continue erecting alternate mould panels until the entire circumference is enclosed.

7.6 Make sure that the shuttering is not on top of the key as this will make it difficult to strip. Note that the size of the shutter can be adjusted by rotating the 75 x 100 timbers between the adjoining angle irons (see point 7.4) thereby adjusting the circumference and hence the diameter.

7.7 To maintain the mould in a vertical position brace the panels from the inside with poles where necessary.

7.8 Cover each joint, where the panels overlap on the outside, with a 100 mm damp proof course strip (or plastic) to prevent the plaster from catching on the overlap. Figure 9 below shows the inside of the wall moulds with the poles used for providing support visible. Note the overlap of the panels and the use of plastic sheeting to cover the joints between the panels in the foreground.

Figure 9. View of the inside of the wall moulds showing the use of poles to brace the panels

8. Fixing the Wall Reinforcement

8.1 Cut the weldmesh Ref 156 (3.6 mm diameter with 100 mm squares). The lengths in metres required for each layer can be calculated using the formula below:

Length = (π x Mould Diameter) + 0.5 m

See Appendix A, Table 1 for the mould radii for various sizes of ferrocement reservoir.

8.2 Insert the first layer of mesh between the wall starter bars and the mould and pull the mesh up tight against the mould. It is very important to get the first layer of mesh as close to the moulds as possible. Any space left here will mean that the first coat of plaster has to be made thicker. This makes it difficult to compact the first coat and ultimately causes voids in the wall which are likely to leak. This first layer of mesh will extend 0.5 m above the mould. The extra mesh protruding above the wall shuttering will later overlap with the roof reinforcement.

8.3 Tie the second layer of mesh (where specified) as it is pulled around the mould, otherwise it will not lie flat against the first layer (important for the same reason as in 8.2 above). Cut the portion of mesh extending above the top of the mould off. Note that not all reservoirs require the same number of layers of weld mesh. See Table 2 in Appendix A for values of γ i.e. the number of layers of ref 156 weld mesh required for various sizes of ferrocement reservoir.

8.4 Tie the vertical wall bars at equal spacing around the mesh (at every second wall starter bar). Note that the vertical wall bars are only required on reservoirs of 15 kℓ or greater.

8.5 Wind γ hoops of the hard drawn ungalvanised wire (3.15 mm diameter) tightly around the moulds along each corrugation in the panels. δ is the number of hoops required per mould corrugation and is based on the size of the reservoir. See Table 2 in Appendix A for values of δ for various sizes of ferrocement reservoir. Tie the hard drawn wire to the weld mesh.

8.6 Tie the ring reinforcement to the first layer of mesh above the top of the second layer of mesh at the level of the mould. Note that the ring reinforcement is only used on reservoirs of greater than 15 kℓ in size.

8.7 Finally, tie a layer of 1800 mm high fowl netting (50 x 50 mm) to the weld mesh. The netting helps to reduce micro-cracking and helps to keep the mortar in place as it is worked into the mesh. Figure 10 shows a view of a 71 kℓ reservoir with all wall reinforcement in place.

Figure 10. The wall reinforcement in place on a 71 kℓ ferrocement reservoir

9. Mortar Mix

9.1 The wall is built up by applying two coats of plaster onto the outside of the mould, then removing the mould and applying two coats onto the inside.

9.2 The mortar mix used is 1 part cement:3 parts river sand. If the river sand contains insufficient fines, blend it with plastering sand. The proportions of this blend depend on the roughness of the river sand; usually a blend ratio of 2 parts river sand to 1 part plastering sand will be sufficient. For very coarse river sands such as Umgeni sand, a blend ratio of 1:1 is recommended. The fines help to seal the tank although mortar made with sand containing too much fines is prone to cracking.

9.3 A dryer, stiffer mix will be stronger than a wet mix assuming that the compaction is the same. However it is difficult to work a stiff mix into the formwork. A wetter mix is much easier to work with but the resulting wall will be more permeable to water, have a lower strength and be more prone to shrinkage cracks. The mortar must be creamy and workable so that it can penetrate the mesh without sagging. The cement water ratio (by weight) should not be allowed to exceed 2:1 (i.e. one 50 kg bag of cement to 25 litres of water). A concrete mixer must never be used to mix the mortar because the mortar has a tendency to stick to the drum of the mixer. To prevent this from occurring, workers tend to add excessive water which weakens the mix.

9.4 A batch of mortar should ideally be used up within 30 minutes as it becomes progressively unworkable without adding extra water. Under no circumstances should mortar that has begun to harden be used. Adding water to an old batch of mortar should never be allowed as this weakens the mix.

9.5 By paying due attention to the mix proportion of the mortar and with proper curing it is possible to produce a mortar mix with a 28 day strength in excess of 30 MPa, which is in excess of design requirements.

10 Applying the First Outside Coat

10.1 Clean the base and key outside of the mould and ensure that there is no loose material in the key that could cause leaks. The key should be wetted taking care not to leave any pools of water.

10.2 Trowel the mortar onto the mould working from the base upwards and push it firmly through the mesh to fill the corrugations so that it just covers the reinforcing. The fowl netting should still just be visible. Plaster in vertical sections. By starting at the base and working up, the plaster is always supported from below which helps to avoid slumping. Make sure that the key is filled during the application of the first coat. Figure 11 below shows the first outside coat being applied. Note that the fowl netting is only just being covered by the mortar.

10.3 After each vertical section is completed, the mortar must be covered by a strip of plastic or wetted sacking to prevent it from drying out and cracking. Figure 12 below shows an example of a reservoir that has been wrapped in plastic to assist with the curing process.

Figure 11. Application of the first outside coat

10.4 The plastic or wet sacking should be left covering the mortar until the next coat is applied. Wet the wall two or three times per day.

10.5 After the coat has been completed, its surface should be roughened to promote bonding with the next coat of mortar.

Figure 12. Use of plastic to assist with the curing of a reservoir wall

11. The Second Outside Coat

11.1 Apply the second outside coat in the same manner as the first coat. This coat is however much thinner than the first coat (between 10 and 15 mm thick). As in section 10.1 above, clean and wet the base again before plastering begins. Cover the vertical sections of plaster as they are completed (see sections 10.3 and 10.4 above).

11.2 Note that at the junction between the base and the wall, a mortar collar is created by thickening the wall. An example of this can be seen in Figure 13 below.

11.3 It is possible to do more than one coat in a day. To avoid slumping on the successive coat the plasterers should start working at one end and move around the tank in one direction. This allows the plaster at the start point time to set before the next coat is applied. Apply the following coat in the same manner starting at the same point

11.4 The second and final coat is finished with a steel float.

Figure 13. Thickening of the wall above the base to form a mortar collar

11.5 To ensure that a good bond is achieved between layers the standing period between coats should not be more than one day. Figure 14 shows a 120 kℓ reservoir with the second outside coat completed.

Figure 14. A 120 kℓ reservoir with outside coats completed

12. Removing the Wall Shuttering

12.1 Remove the poles propping up the panels and pull out the timbers between the angle irons.

12.2 Pull the panels away from the wall by hand, ensuring that no leverage is placed on the wall as the plaster is still relatively "green".

12.3 Lift the panels over the wall clear of the tank. *Do not lean ladders against the wall.*

12.4 Clean the panels thoroughly of any mortar or cement. After cleaning, put a film of shutter oil (or old motor oil) onto the corrugated iron.

13. Applying the First Inside Coat

13.1 Remove all the loose mortar from the inside of the wall, especially where honeycombing has occurred.

Figure 15. A view of the inside of a reservoir wall before the casting of the first inside coat

13.2 Remove the plastic strips where the panels overlapped.

13.3 Where the plaster of the inside wall is smooth, roughen it up with a steel brush.

13.4 Remove all dry mortar, pieces of wire etc. from inside the tank, paying special attention to the joint between the base and wall. Figure 15 above shows the inside of a reservoir wall before casting of the first inside coat begins.

13.5 Wet the inside of the wall and the base well but do not allow pools of water to form.

13.6 Erect scaffolding on both the inside and outside of the tank. The scaffolding must be safe for the workers to stand on and handle the buckets filled with mortar.

13.7 To facilitate the passing of buckets of mortar over the wall, the weld mesh can be cut vertically and bent towards the outside.

13.8 Complete the plastering as in section 10 above.

13.9 Fill all the corrugations left by the mould with the first coat taking care not to make this layer too thick. Figure 16 below shows the inside of a reservoir after the first inside coat has been completed.

Figure 16. A view of the inside of a reservoir wall after the first inside coat has been cast

14. The Second Inside Coat

14.1 Apply the second inside coat as in section 11 above. Figure 17 below shows the inside of a reservoir wall after the second inside coat has been completed.

Figure 17. A view of the inside of a reservoir wall after the second inside coat has been cast

15. Erecting the Shuttering and Reinforcing for the Roof

Four types of roof shuttering are used: web bars, radial bars, starter bars and hoops. The web bars connect directly to the radial web in the centre of the roof. The radial bars move outwards from the web bars and provide a connection between the web bars and the starter bars. The starter bars are the outermost shuttering radials and are designed to rest on top of 1.8 m poles placed next to the wall of the reservoir. The hoops are flat, flexible pieces of steel used to support the radial bars. The arrangement of these elements is shown in Figure 18. Figure 20 shows how the starter bars rest on top of the 1.8 m poles positioned next to the reservoir wall.

Figure 18. Roof shuttering showing the hoops as well as the web, radial and starter bars

15.1 Use hard hats at all times because a falling piece of roof shuttering can cause serious injury.

15.2 Locate the centre of the reservoir using the tape measure and make a mark on the base.

15.3 Obtain a centre pole. The length of the pole will depend on the radius of the reservoir being built. Note that the roof is designed to slope at 17°, and if it is built flatter than this the stresses in the roof will be greater than the design stress. See Table 1 of Appendix A for centre pole heights for various sizes of ferrocement reservoirs. Also obtain four 25×100×400 mm planks Nail these planks parallel to the pole flush with one end of the pole.

15.4 Attach the radial web to the side of the pole where the planks are nailed.

15.5 Erect a scaffold at the centre of the reservoir, high enough for an average person to reach the top of the roof.

15.6 Hold the centre pole up and install four complete sets of roof shuttering radial supports on opposing sides from one another i.e. at angles of 0, 90, 180 and 270 degrees if a plan view of the roof is taken. Figure 19 shows the positioning of the centre pole of a 47 kℓ reservoir.

Figure 19. Positioning of the centre pole of a 47 kℓ reservoir

15.7 Use the spirit level to make sure the centre pole is vertical. Tie the four starter bars to the 1.8 m poles and leave the layer of weld mesh overlapping.

15.8 Fit all web bars to the radial web. Join these with 1 m radial bars (with a slightly larger diameter than the radial web bars). Leave a space of ½ m from the radial web and position hoop one. Thereafter hoops are installed at 1 m intervals and the number of hoops depends on the diameter of the tank. For example, a 120 kℓ reservoir with a radius of 4.85 m would use five sets of hoops. Allow a space of ½ m between the final hoop and the reservoir wall.

Figure 20. Supporting of the radial roof shutter struts using wooden poles

15.9 Every hoop is to be supported with either creosote or steel poles. Retractable steel poles are preferable so that they can be lengthened / shortened to obtain the correct roof shape. The poles should not be exactly vertical but perpendicular to the roof (see Figure 21) and should be tied to the radial hoops so that they do not fall when the other supports are moved. The heights of each set of hoops must be adjusted until the shape created by the shuttering is completely concave. If the shape of the shuttering is even slightly convex, the strength of the roof is greatly decreased. Figure 21 over the page shows the underside view of the completed roof shuttering.

15.10 Once the shape of the shuttering is sufficiently concave, flat steel sheets are placed over the shuttering. The sheets should be painted with mould oil and holes punched in them before being positioned on the roof. This allows for the sheets to be tied to the shuttering and prevents sagging after the roof has been cast. Figure 22 shows how the flat sheets are positioned on top of the roof shuttering.

15.11 Cover the roof shuttering with a layer of fowl netting (for reservoirs up to 15 kℓ in size) or galvanized fencing mesh (for larger reservoirs). Fold on top of this the 0.5 metre section of Ref 156 mesh left over from the wall (for reservoirs larger than 15 kℓ in size) then place bars of Y12 hoop reinforcement on top of this mesh around the circumference of the reservoir. The first ring should be positioned 100 mm from the roof/wall joint with additional rings being placed inside the first ring at a spacing of 100 mm. The number of rings (ε) depends on the size of the reservoir (See Table 2 in Appendix A for values of ε for various sizes of ferrocement reservoir). The ring reinforcing should be tied to the fencing mesh / fowl netting. Figure 23 provides a close up view of the ring reinforcing used on a 200 kℓ reservoir.

Figure 21. Underside of roof shuttering showing the method of supporting the hoops using retractable steel poles

Figure 22. Positioning of the flat steel sheets

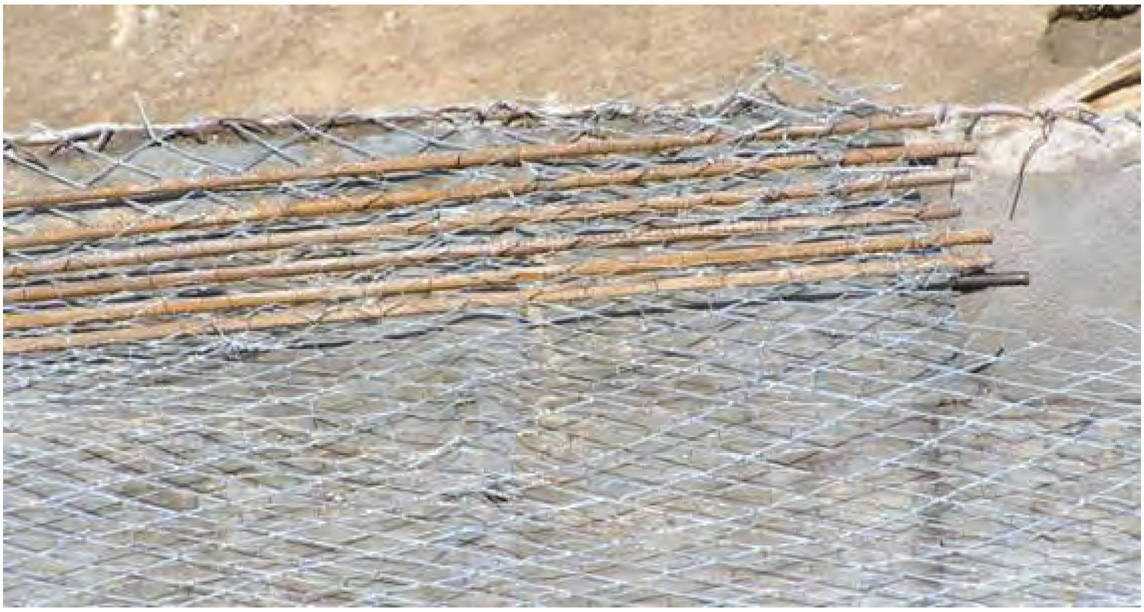


Figure 23. Roof hoop ring reinforcing on a 200 kℓ reservoir

15.12 A second layer of galvanised fencing mesh should be placed around the outermost 2 m of the roof and tied to the lower layer.

15.13 A 600 x 600 mm space should be cut into the mesh for the installation of the manhole hatch. The hatch should be positioned in close proximity to where the inlet pipe comes through the wall of the reservoir to allow for maintenance of the float valve from the hatch. The frame for the hatch must be tied to the galvanized fencing mesh. A separate access hole should also be left open without steel sheeting, mesh and reinforcement in order to allow for access to the reservoir for the purposes of removing the roof shuttering.

15.14 Note that the setting up of the roof shuttering for reservoirs of 5 to 15 kℓ in size differs in that the hatch is positioned at the centre of the reservoir roof. Hence, the shuttering used for the roofs on these reservoirs is different and consists simply of wooden poles set along the wall as well as underneath the hatch itself. The poles are secured together by nailing planks between the poles and the flat steel sheets are then placed on top of these planks. The reinforcement for the roofs of these reservoirs is also different as only fowl netting (i.e. no galvanized fencing mesh or ring reinforcing) is used.

15.15 Install the tank inlet pipework.

16. Casting the Roof

16.1 Erect scaffolding so that the mortar can be lifted onto the roof.

16.2 Remove any loose mortar at the top of the wall and wet the roof shuttering.

16.3 Use a 1:3 mortar mix using river sand (2 bags of cement:3 wheelbarrows of river sand). The mortar should be slightly drier than that used for the walls.

16.4 Begin plastering in circular sections from the centre point of the roof. The plastering is usually done in two layers. The first layer is applied such that it only just covers the top layer of galvanised fencing mesh. Move outwards from the centre in circular sections until the entire roof is covered. The second coat of mortar can follow shortly behind the first coat and should be cast in the same manner. However, the second coat must ensure that a minimum cover of 15 mm to all reinforcement is achieved. The second coat should in addition be floated to achieve a smooth finish. Figures 24, 25 and 26 show the casting of a 220 kℓ reservoir roof at various stages.

Figure 24. Casting of the first coat begins

Figure 25. Casting of the first coat is well underway and casting of the second coat has begun

Figure 26 The first coat has been cast while the second coat is nearing completion

16.5 If possible cast the roof in one day. If the roof is not completed in one day, remove all loose mortar from the working joints and apply a thin coat of cement slush before resuming.

16.6 When placing the mortar around the access hole described in section 15.14 above, allow the mesh to protrude 200 mm from the edge of the mortar so that a proper overlap can be achieved when the access hole is closed.

16.7 Cure the roof for a minimum of 7 days. The recommended method of curing is to cover the roof with plastic (masking tape can be used to secure the plastic). Figure 27 shows the same 220 kℓ reservoir after sufficient curing has taken place and the plastic on the roof is being removed. Note the access hole used during the erection and dismantling of the shuttering is visible on the right.

Figure 26. The 220 kℓ reservoir roof after curing with the plastic being removed

Important Note: *The roof is designed as a thin shell convex structure. If it is neither thin in section (60 mm or less) nor convex in shape, it may be subject to higher stresses than it can withstand. Workers have a tendency to make up for poor shape control during the shutter erection stage by using extra mortar to make up the roof shape to the right outward appearance. This may conceal thick sections which are convex on the outside, but locally concave underneath. This is dangerous, creating stresses in the roof for which it is not designed. A competent person must therefore inspect and approve the shape of the roof shuttering before the mortaring of the roof commences.*

17. Removing the Roof Shuttering

17.1 Only remove the shuttering after the roof has been allowed to set for at least seven days.

17.2 Start by removing the first three planks closest to the access hole. Strip all the sheets within that segment. These sheets and planks can be removed from the inside of the tank.

17.3 When only the centre pole is left standing, all but two workers should exit the reservoir. Scaffolding should be erected around the centre pole to provide backup protection while the centre pole is removed. After the centre pole is removed and the roof is supporting its own weight, the scaffolding can be removed.

17.3 Clean the steel sheets, removing the cement and mortar. Remove all nails from the planks and poles.

17.4 Place shuttering under the access hole and fix the reinforcement as described in section 15. Close the access hole with mortar.

17.5 If any reinforcing on the underside of the roof is not adequately covered, these areas should be patched.

18. Tasks and Labour Requirements

Table 3 in Appendix A contains suggested team sizes for various sizes of ferrocement reservoirs. Note that these figures are estimated and may differ depending on site conditions. These figures are also for site staff only and do not include a construction manager. The construction manager should carry out weekly inspections in order to provide quality control assurance as well as to monitor the progress of the construction team. The number of team members should be adjusted if it is found that construction is falling behind programme.

The length of time worked will vary depending on the task for the day. For example, casting a base may take the whole day, depending on the size of the reservoir, whereas fixing the reinforcement for the base may only require a morning. Care should therefore be taken when setting out tasks to ensure that optimal use of the work force is made.

19. Work Programme

Table 4 in Appendix A contains a work programme detailing the number of days required to complete the required construction activities for various sizes of ferrocement reservoirs.

20. Tools and Equipment Requirements

The following list of tools will be required by the team for a medium sized reservoir (e.g. 70 kℓ):

a)	Steel floats	4
b)	Wooden floats	4
c)	Hand hawks	4
d)	Steel fixing pliers	4
e)	Pliers	1
f)	Tin snips	1
g)	Level	1
h)	Hacksaw	1
i)	Hammer 2lb	1
j)	Chisel	1
k)	Bolt cutters	1
l)	5 m tape measure	1
m)	50 m tape measure	1
n)	Floating aids	4
o)	Scaffolding trestles	4
p)	Base depth gauges	4
q)	Wire brushes	4
r)	Builders brushes	4
s)	Builders buckets	4
t)	Wheelbarrows	4

21. General Factors to Consider

In planning for a new job the construction manager must consider the following factors.

21.1 Labour:

a) The number of artisans and labourers required?
b) Schedule of payments: daily, weekly or contractual?
c) Qualifications of the artisans?
d) Motivation of the workers?
e) Qualifications of the team leader?
f) Effective hours put in per day or per week?
g) Do workers have to depend on transport?

21.2 Reservoirs:

a) The number of reservoirs to be constructed?
b) The proximity of the reservoirs to each other?

21.3 Crime

a) Can material and equipment be stored on site or does it have to be moved every day to a safe place?
b) How close to the reservoir site/s is the place that can provide safe storage for materials and equipment?

21.4 Materials and Equipment

a) How promptly can cement, sand and stone be delivered to the site?
b) Is there water readily available?
c) Are there proper and sufficient tools?

21.5 Management and Supervision

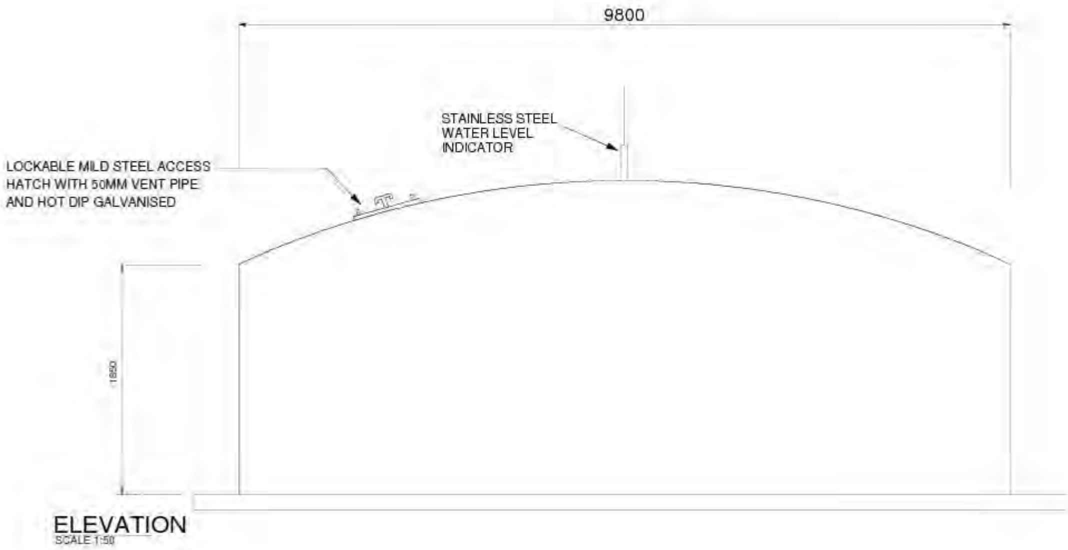
a) Is there good communication between the construction manager and the construction team?
b) Is it frequent and regular?

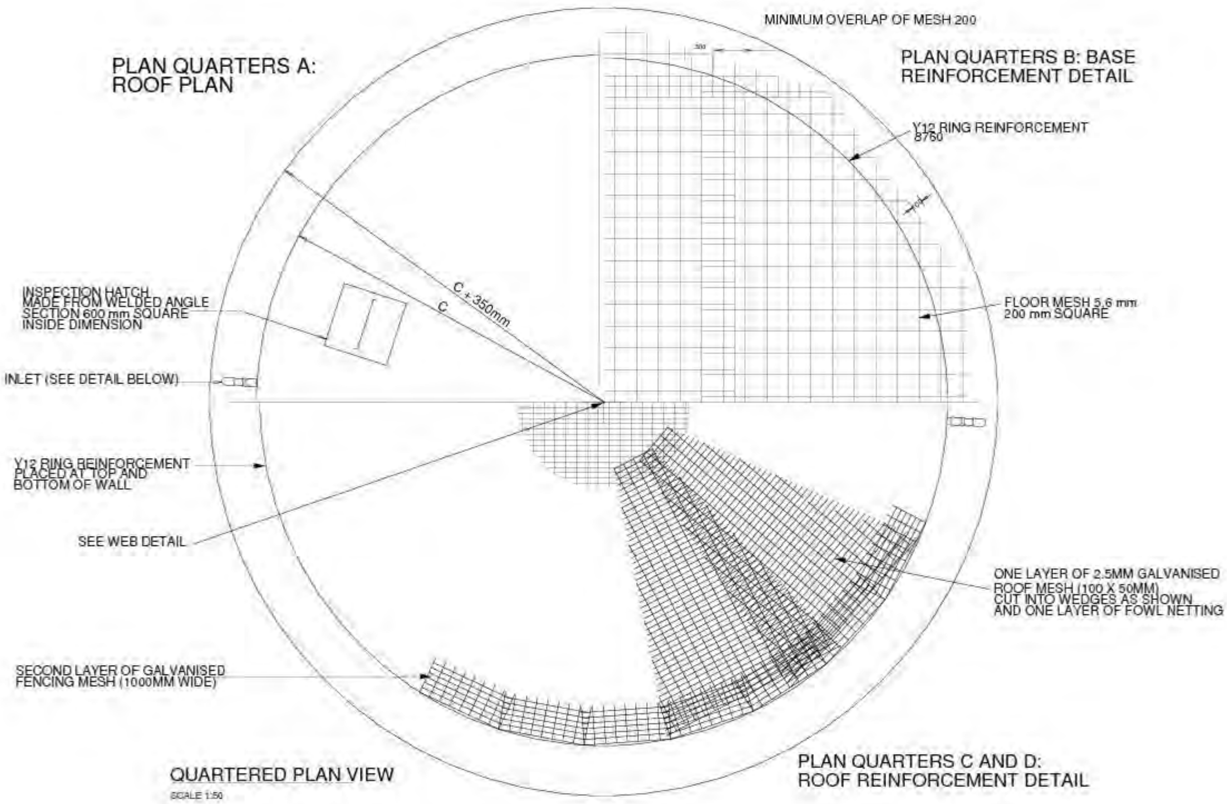
21.6 Safety

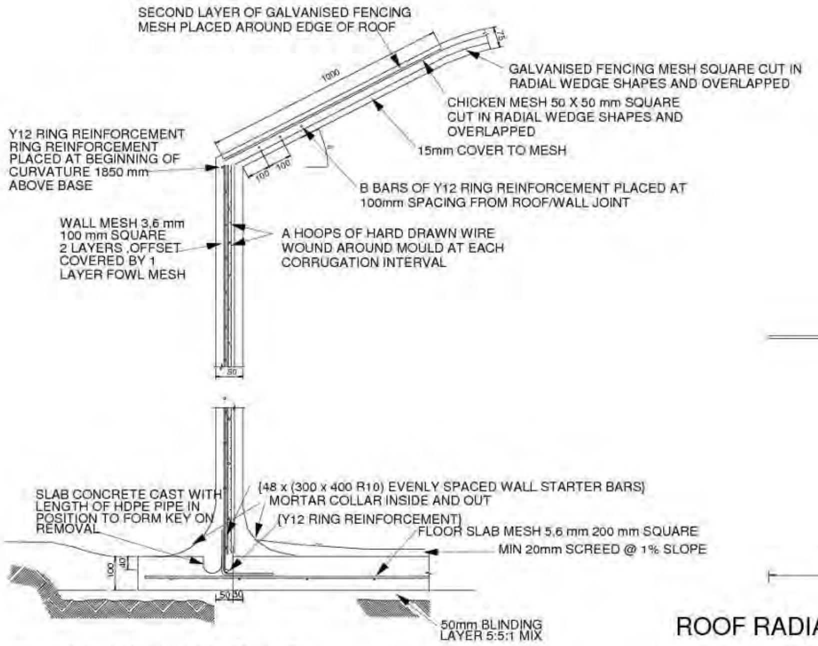
a) Do all the workers have the correct protective wear (boots, hard hats, gloves)?
b) Are the scaffolds and ladders sturdy and safe?
d) Has a safety file been compiled for the job in accordance with the OHSA?
e) Have temporary workers had a safety induction and is there proof of this on file?

Appendix A

Extracts from drawing of a 120 kℓ Ferrocement Reservoir

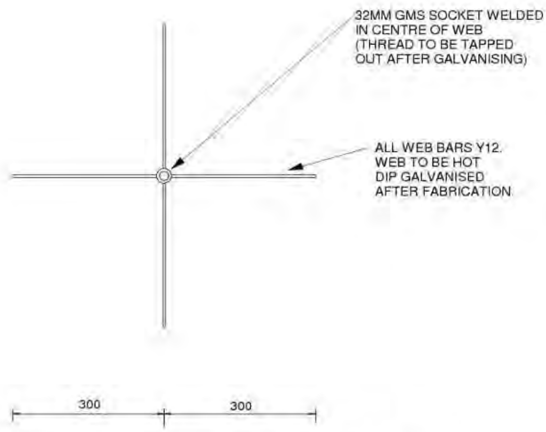

9800
STAINLESS STEEL WATER LEVEL INDICATOR
LOCKABLE MILD STEEL ACCESS HATCH WITH 50MM VENT PIPE AND HOT DIP GALVANISED
1850
ELEVATION
SCALE 1:50


MINIMUM OVERLAP OF MESH 200
PLAN QUARTERS A: ROOF PLAN
PLAN QUARTERS B: BASE REINFORCEMENT DETAIL
Y12 RING REINFORCEMENT 8750
INSPECTION HATCH MADE FROM WELDED ANGLE SECTION 600 mm SQUARE INSIDE DIMENSION
C + 350mm
C
FLOOR MESH 5.6 mm 200 mm SQUARE
INLET (SEE DETAIL BELOW)
Y12 RING REINFORCEMENT PLACED AT TOP AND BOTTOM OF WALL
SEE WEB DETAIL
ONE LAYER OF 2.5MM GALVANISED ROOF MESH (100 X 50MM) CUT INTO WEDGES AS SHOWN AND ONE LAYER OF FOWL NETTING
SECOND LAYER OF GALVANISED FENCING MESH (1000MM WIDE)
QUARTERED PLAN VIEW
SCALE 1:50
PLAN QUARTERS C AND D: ROOF REINFORCEMENT DETAIL



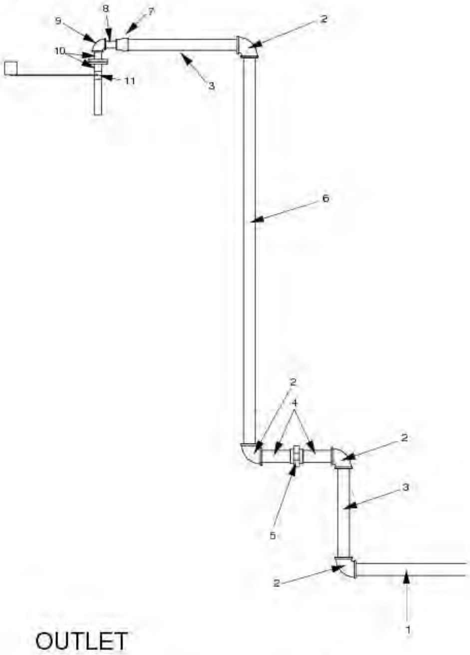
BASE - WALL JOINT AND WALL - ROOF DETAIL

N.T.S.



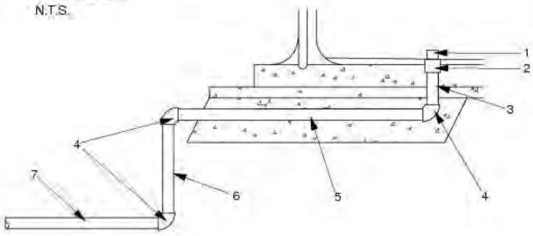
ROOF RADIAL REINFORCEMENT WEB DETAIL

INLET
N.T.S.



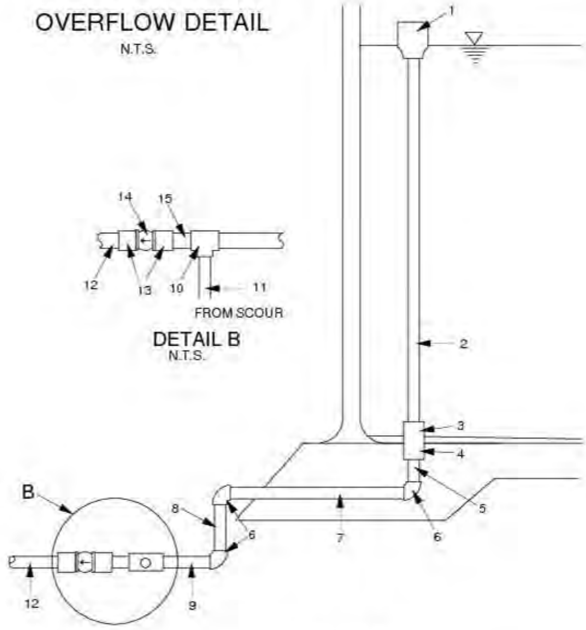
KEY: INLET		
ITEM	DESCRIPTION	QUANTITY
1)	r mm G.M.S. PIPE FROM METER CHAMBER	1
2)	r mm G.M.S. 90 DEGREE THREADED ELBOW	4
3)	r mm G.M.S. PIPE CUT AND THREADED 400MM LONG	2
4)	r mm G.M.S. PIPE CUT AND THREADED 150MM LONG	2
5)	r mm G.M.S. UNION	1
6)	r mm G.M.S. PIPE CUT AND THREADED 1700MM LONG	1
7)	r x 25mm G.M.S. REDUCING SOCKET	1
8)	25mm G.M.S. BARREL NIPPLE	1
9)	25mm G.M.S. 90 DEGREE THREADED ELBOW	1
10)	25mm G.M.S. NIPPLE 60mm LONG THREADED AND FLANGED ON OPPOSITE ENDS	2
11)	25mm BALEM STAINLESS STEEL FLOAT VALVE (INCL. SYPHON PIPE)	1
NOTES		
1)	ALL SCREWED JOINTS TO BE MADE WITH STAG AND HEMP.	

OUTLET
N.T.S.



KEY: OUTLET		
ITEM	DESCRIPTION	QUANTITY
1)	n mm MPVC CLASS 12 EXACT 100 MM LONG (REMOVABLE)	1
2)	n mm SOCKET, MPVC, SOLVENT CEMENT	1
3)	n mm MPVC CLASS 12 NIPPLE	1
4)	n mm 90 DEGREE ELBOW, MPVC, SOLVENT CEMENT	3
5)	n mm MPVC CLASS 12 APPROX 2500MM LONG	1
6)	n mm MPVC CLASS 12 EXACT 400MM LONG	1
7)	n mm MPVC CLASS 12 TO PUMP STATION / OUTLET CHAMBER	1
NOTES		
1)	ITEM 1 (UPVC EXACT) IS NOT TO BE SOLVENT CEMENTED TO ITEM 2 (UPVC SOCKET). ITEM 1 IS REMOVED DURING THE CLEANING OF THE TANK.	
2)	ITEMS 2 TO 5 ARE TO BE COATED IN SOLVENT CEMENT AND COVERED WITH RIVER SAND TO ENSURE GOOD BONDING TO THE SURROUNDING CONCRETE.	

OVERFLOW DETAIL
N.T.S.



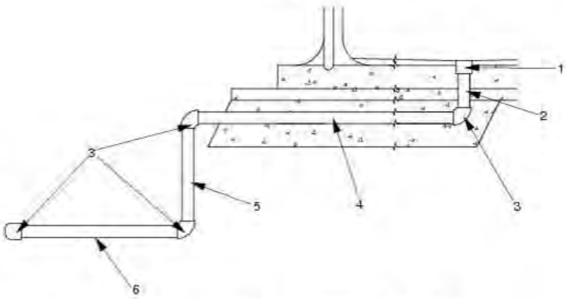
KEY: OVERFLOW

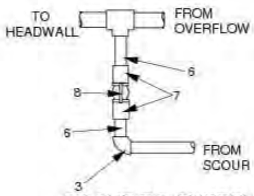
ITEM	DESCRIPTION	QUANTITY
1)	p mm X n mm MPVC REDUCING SOCKET	1
2)	n mm MPVC CLASS 16 EXACT 1700MM	1
3)	n mm SOLVENT CEMENT MALE ADAPTOR	3
4)	n mm SOLVENT CEMENT FEMALE ADAPTOR	1
5)	n mm MPVC CLASS 12 NIPPLE	2
6)	n mm 90 DEGREE ELBOW, SOLVENT CEMENT	3
7)	n mm MPVC CLASS 12 APPROX 1400MM	1
8)	n mm MPVC CLASS 12 EXACT 400MM	1
9)	n mm MPVC CLASS 12 EXACT 500MM	1
10)	n x q mm SOLVENT CEMENT REDUCING TEE PIECE	1
11)	n mm MPVC PIPE FROM SCOUR (BILLED ELSEWHERE)	N/A
12)	n mm MPVC PIPE TO HEADWALL	1
13)	n mm NON RETURN VALVE (VERMIN TRAP)	1

NOTES

1) ALL SCREWED JOINTS TO BE MADE WITH STAG AND HEMP.
2) SIZE p TO BE ONE SIZE GREATER THAN SIZE n.

SCOUR DETAIL
N.T.S.





SCOUR TO OVERFLOW CONNECTION DETAIL (PLAN VIEW)

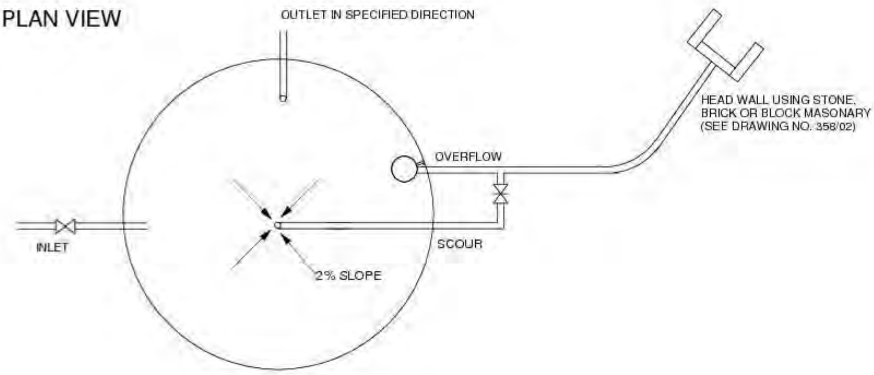
KEY: SCOUR

ITEM	DESCRIPTION	QUANTITY
1)	q mm SOCKET, MPVC, SOLVENT CEMENT	1
2)	q mm MPVC CLASS 12 NIPPLE	1
3)	q mm 90 DEGREE ELBOW, MPVC, SOLVENT CEMENT	4
4)	q mm MPVC CLASS 12 APPROX 4400MM LONG	1
5)	q mm MPVC CLASS 12 EXACT 400MM LONG	1
6)	q mm MPVC CLASS 12 CUT TO SUIT	3
7)	q mm MALE ADAPTOR, MPVC, SOLVENT CEMENT	2
8)	q mm BRASS GLOBE VALVE	1

NOTES

1) ITEMS 2 TO 5 ARE TO BE COATED IN SOLVENT CEMENT AND COVERED WITH RIVER SAND TO ENSURE GOOD BONDING TO THE SURROUNDING CONCRETE.

SCHEMATIC, PLAN VIEW
N.T.S.



Appendix B

Reservoir Dimensions and Reinforcing Details

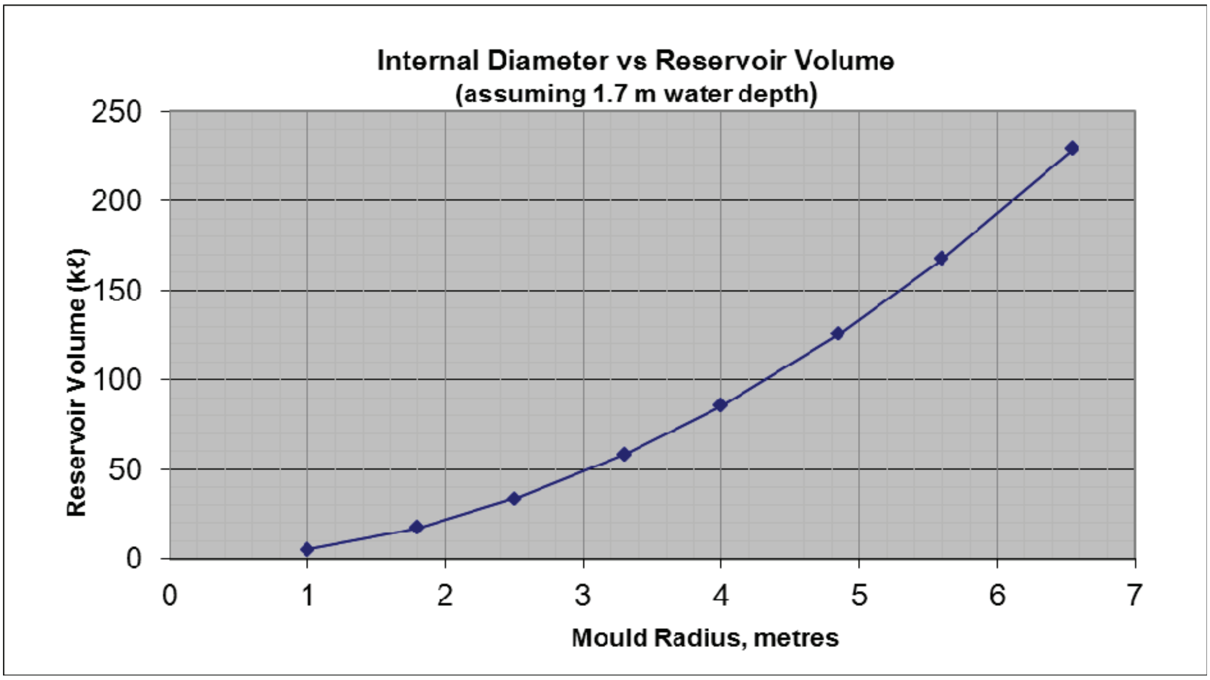
TABLE 1: FERROCEMENT RESERVOIR DIMENSIONS

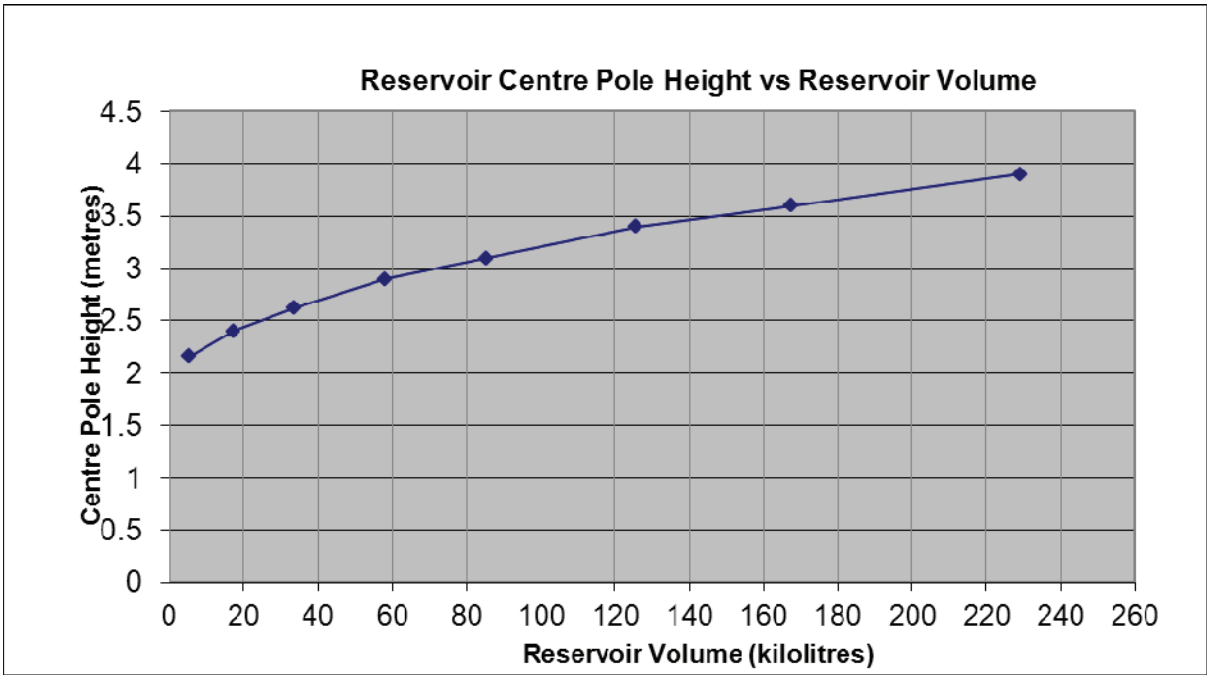
Nominal Reservoir Volume (kl)	Base Radius (m)	Mould Radius (m)	Wall Height (m)	Centre Pole Height (m)
5	1.27	0.92	1.85	2.20
10	1.73	1.38	1.85	2.25
15	2.20	1.72	1.85	2.35
20	2.43	2.01	1.85	2.45
40	3.12	2.77	1.85	2.75
50	3.58	3.12	1.85	2.85
70	4.04	3.69	1.85	3.00
100	4.73	4.38	1.85	3.25
120	5.20	4.85	1.85	3.40
150	5.66	5.31	1.85	3.50
200	6.58	6.23	1.85	3.75

Note: The radius is determined by the number of 1.5 metre mould panels that are used and the degree of overlap between the panels (typically between 50 mm and 150 mm)

TABLE 2: FERROCEMENT RESERVOIR REINFORCING DETAILS

Reservoir Size (kl)	α (No. of wall starter bars)	β (No. of mould panels)	γ (No. of layers of weld mesh)	δ (No. of hoops of HD wire)	ε (No. of hoops of ring reinf.)
5	0	4	1	0	0
10	0	6	1	1	0
15	0	8	1	1	0
20	32	9	1	1	1
40	32	12	2	1	1
50	32	14	2	1	2
70	32	16	2	2	3
100	40	19	2	2	3
120	40	21	2	2	4
150	48	23	2	3	5
200	48	27	2	3	7


Internal Diameter vs Reservoir Volume
(assuming 1.7 m water depth)
Reservoir Volume (kℓ)
250
200
150
100
50
0
0
1
2
3
4
5
6
7
Mould Radius, metres


Reservoir Centre Pole Height vs Reservoir Volume
Centre Pole Height (metres)
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
0
20
40
60
80
100
120
140
160
180
200
220
240
260
Reservoir Volume (kilolitres)

Appendix C

Tables Providing Labour Figures and Work Programmes

TABLE 3: APPROXIMATE LABOUR REQUIREMENTS

Reservoir Size (kl)	No. of Unskilled Labourers	No. of Skilled Labourers	Total
5	3	1	**4**
10	3	1	**4**
15	4	1	**5**
20	4	2	**6**
40	5	2	**7**
50	6	2	**8**
70	6	3	**9**
100	7	4	**11**
120	8	4	**12**
150	10	4	**14**
200	12	5	**17**

TABLE 4: TYPICAL WORK PROGRAMMES

Reservoir Size (kl)	**5**	**10**	**15**	**20**	**40**	**50**	**70**	**100**	**120**	**150**	**200**
Construction Activity	**No. of days required**										
Final Levelling	0.5	1	1	1	1	1	1	1	1	1	1
Casting Blinding	1	1	1	1	1	1	1	1	1	1	1
Setting Up Base Reinforcement	0.5	1	1	1	1	1	1	1	1	1	1
Casting Base	1	1	1	1	1	1	1	1	1	1	1
Setting Up Wall Moulds and Reinforcement	1	1	1	2	2	2	3	3	3	4	5
Casting Outside Coats	2	2	2	2	2	2	2	2	2	2	2
Stripping Wall Moulds and Casting 1st Inside Coat	1	1	1	1	1	1	1	1	1	1	1
Casting 2nd Inside Coat	1	1	1	1	1	1	1	1	1	1	1
Setting Up Roof Shuttering and Reinforcement	1	1	1	2	3	3	3	4	4	4	5
Casting Roof	1	1	1	1	1	1	1	1	1	1	1
Stripping Roof, Cleaning and Bagwashing Reservoir	1	1	1	1	1	1	1	1	1	1	1
Contingencies	1	1	1	1	1	1	1	1	1	1	2
Total	**12**	**13**	**13**	**15**	**16**	**16**	**17**	**18**	**18**	**19**	**22**