Service reservoirs of the Rand Water Board

F S WHITEHEAD* (Member) and R J LABURN** (Fellow)

Synopsis

The function of service reservoirs in the pumping and distribution systems of a bulk water supply undertaking and its consumers is described. The evolution and development since 1903 of the arrangement of service reservoirs for the Rand Water Board is set out and desirable design features and construction procedures listed.

Samenvatting

Die funksie van diensreservoire in die pomp- en distribuieystelsels van 'n onderneming wat water by die groot maat verskaf en van sy verbruikers word beskryf. Die evolusie en ontwikkeling van diensreservoire vir die Randwaterraad sedert 1903 asook die gewenste ontwerphienerke en konstruksieprocedures word uiteengesit.

Introduction:

The Rand Water Board’s main area of supply straddles the Witwatersrand watershed between the south, which drains towards the Atlantic Ocean, and the north flowing rivers which find their way to the Indian Ocean via the Limpopo. Originally water for consumption in the Witwatersrand gold mining settlements was supplied from small local springs and wells, but it soon became necessary to look southwards to the lower-lying dolomite areas for supplementary supplies. This established the configuration of the Board’s bulk supply system which comprises pumping systems made up of long pumping mains supplying consumers situated on an elevated area whose requirements are drawn off mainly during the daylight hours.

The configuration and the topography have made it difficult and uneconomic to provide large capacity reservoirs or dams storing purified water on the Witwatersrand for the purpose of obtaining security of supply for any length of time because of the lack of suitable sites and the prohibitive cost of covered clear-water reservoirs. This has led to the water supply being essentially a pumping system supplying direct to consumers. Up to about 1941 provision was made for only such storage as was necessary to spread the pumping load over the greater part of the 24 hours of the day. Security of supply was obtained by installing 100 per cent stand-by plant and duplicate methods of supplying areas in an emergency.

The first reservoir to be provided for Johannesburg had a capacity of 3, 6 Ml and was constructed in 1888 in Berea Township near the corner of Abel and Harrow Roads. Subsequently additional reservoirs were constructed to serve the developing township including a reservoir with a capacity of 17 Ml at Yeoville and several high-level steel water tanks at various places.

The establishment of the Board in 1903 led to its taking over the old Berea and Yeoville reservoirs from the water supply companies in April 1905 and these together with the local reticulation were immediately transferred to Johannesburg Municipality. Under the Board’s Klip River Valley Scheme, which was brought into service in 1908, a reservoir having a capacity of 5,8 Ml was erected at Turffontein Nek and another with a similar capacity at the Village Main pumping station. As the supply extended eastwards a reservoir was built at Simmer & Jack and another at Leeuwyno. Towards the west the Board, in 1907, decided to construct a reservoir at Brixton Hill to improve the efficiency of operation of the Paarlhoop pumping plant.

Initially the capacity of Brixton reservoir was to be 1,82 Ml and it was anticipated that the contents would be for the use of Johannesburg municipality. The municipality, however, arranged with the Board that the capacity be increased to 5,8 Ml and agreed to pay half the construction costs. In 1923, when the Board rearranged its central distribution system it transferred its interest in Brixton reservoir, inclu-

F S Whitehead obtained a BSc degree in Civil Engineering at the Witwatersrand University in 1943. Following service in the Royal Navy, he worked in the Public Works Department in Pretoria for two years and thereafter for various contractors and consulting engineers until he joined the Rand Water Board in 1963. He was appointed civil engineer in 1968 and an Assistant Chief Engineer in 1978.

R J Laburn obtained a BSc degree in Civil Engineering at the University of the Witwatersrand in 1940 and an MSc (Eng) degree at the same university in 1942. After military service with the South African Engineering Corps and the Royal Engineers he joined the Rand Water Board in 1946 and in 1965 was appointed its Chief Engineer. He retired in March 1979 and joined a firm of consulting civil engineers. In 1980 the University of the Witwatersrand conferred on him the degree of Doctor of Engineering for his treatise entitled The Rand Water Board — its role and achievements as a regional water supply authority. He is a Member of Council and past president of the Institution, the South African representative of the International Water Supply Association, a member of its Scientific and Technical Council and Chairman of the IWASA International Standing Committee on Water Distribution. He is a member of the South African National Water Research Council and the author of numerous papers on water supply and associated subjects.

Further westwards a small reservoir (capacity 0,8 Ml) was erected at Roodpoort in 1907 and another (capacity 6,3 Ml) was erected at Krugersdorp in 1909. The reservoirs at Turffontein, Village Main, Simmer & Jack, Leeuwyno, Roodpoort and Krugersdorp with a combined capacity of 32,5 Ml were provided by the Board to control and distribute efficiently the supply which in 1910 averaged approximately 32 Ml/d, the area of supply extending at that time from Randfontein to Benoni.

In 1920 increased consumption in the eastern areas had made it necessary to provide a reservoir with a capacity of 11,4 Ml on the high ground north of Germiston at Signal Hill and another on the high ground north of Benoni. These reservoirs served as the respective terminal points of the pumping mains from the Village Main pumping station and from a temporary pumping station which obtained its water from the gold mines in the Springs area. In 1920 the water disposed of by the Board averaged 51,4 Ml/d and the total capacity of the Board’s control reservoirs on the Witwatersrand was 58 Ml.

In 1923 the existing supply from the Klip River Valley was supplemented from the Vaal River on which the Board had constructed a 62

Fig 1: Brixton 5,8 Ml reservoir under construction (1907)
million m³ capacity barrage reservoir situated at an elevation 375 m below the water consuming areas for the purpose of storing untreated water. The system provided for water from the barrage reservoir to be purified and pumped through 40 km of pumping main to Zwarkkopjes pumping station, which boosted delivery to the Witwatersrand. By this time the Turffontein reservoir with its capacity of 5,6 M³ was far too small for the purpose of controlling the supply from Zwarkkopjes pumping station and a new reservoir with a capacity of 23 M³ was provided on the highest available ground, at Forest Hill. This reservoir was supplemented in 1935, to meet further growth, by an additional 90 M³ capacity reservoir and again in 1945 when a 114 M³ capacity reservoir was added.

During the years that followed further reservoirs were constructed for the primary purpose of balancing out fluctuating draw-offs and steadying the pressures in the distribution system during the daily peak period. By 1938 the Board’s total storage on the Witwatersrand was 215 M³ and peak day demands exceeding 260 M³ were imminent. Furthermore, with the rapid growth in demand it was becoming uneconomic to provide 100 per cent plant standby and due to the widespread supply area only limited quantities of water could be transferred between zones in an emergency. As a result it became necessary to recommend that each consumer pay special regard to providing its own storage to enable it to control its own reticulation and to maintain the supply for limited periods in an emergency.

In 1941 further consideration was given to the storage capacity required for efficient operation and it was accepted that the pumping system should be designed to carry the average demand during the highest month and that sufficient reservoir capacity be provided to meet the difference between peak and average demands. From the characteristics of the supply it was established that the storage necessary to balance fluctuating demand during the highest month was capacity equivalent to 30 hours’ normal pumping.

In view of the importance of the area it was also pointed out that additional storage should be provided to meet breakdown and repairs to pipelines and the Board resolved that six hours’ storage should be provided which would allow for 25 per cent of the plant or pipelines being out of service for up to one day. This led to the adoption of the policy that the total capacity to be provided within the area of supply, to permit economic design, efficient operation and security against minor accidents and breakdown, should be equivalent to the expected 36 hours’ average demand during the highest month under normal rainfall conditions.

Just over 10 years ago it was found that the ratio between the average demand during the maximum seven-day period and the average annual demand was showing a rising trend to more than 120 per cent. This was ascribed to the change in relationship between consumption for domestic purposes and consumption for industrial and mining purposes and was further influenced by the building boom in residential townships and a steady improvement in living standards brought about by increased salaries and wages. A close examination of the distribution system of local authorities revealed that in most cases no storage was available to balance the effect of draw-off during peak periods and the Board’s bulk system was being called on to supply at peak rates.

This led to the adoption by the Board in 1970 of the following policy in connection with the supply of water from the gravity pipelines in its distribution system:

1. That the Board continue to provide total storage in its system equivalent to the expected 36 hours’ average demand during the highest month under normal conditions.
2. The Board’s system be designed and developed to meet the maximum day demand of all consumers provided such demand be drawn off at a uniform rate over at least 20 hours of each day.
3. That a consumer be required to design its own system so that peaks in excess of the average demand during the highest day be met by the provision of suitable storage.
4. Where it is not feasible for a particular consumer to provide storage and it becomes necessary to increase the capacity of the Board’s mains and reservoirs to meet the consumer’s peak demands, the cost of such additional capacity be borne by the particular consumer.
5. The number of supply points granted to consumers be limited.

Fig 2: Brixton reservoir nearing completion (1908)

Each consumer drawing a supply from a gravity pipeline in the Board’s system is required to comply with the policy when a new supply point is granted or when an existing connection and meter is to be increased in size and this has led to the provision of reservoir storage by the majority of local authorities.

This policy has recently been reviewed in order to establish the optimum relationship between pumping plant and pipeline and reservoir capacities within the Board’s system. A computer-based model has been developed for simulating the present and future transfer of water in bulk with the prime objective of establishing an optimum relationship between pumping and reservoir storage capacities and operating strategy. From the results it was concluded that:

1. Whichever operating strategy is adopted, the optimum pumping plant and pipeline capacities are equivalent to 98 per cent of the maximum seven-day demand. Current policy is to provide capacity equivalent to 100 per cent of the maximum seven-day demand so the study suggests no significant change.
2. To minimize day-to-day fluctuations in pumping rates, the total capacity of the service reservoirs should be increased marginally to the equivalent of approximately 160 per cent of maximum-month demand. Current policy is to provide 150 per cent of maximum-month demand — again only a small difference.
3. If the system is operated in such a way as to transfer the demand fluctuations from the reservoirs to the pumping plant, however, the required reservoir capacity can be reduced to the equivalent of approximately 137 per cent of maximum-month demand.
4. Operation of the system under condition (3) would marginally increase the energy costs of pumping but the increase would be adequately compensated by savings in capital expenditure on reservoir construction.
5. Should the Board change from the current operating strategy to that described in (3), there would be an immediate reduction in the rate of expenditure on reservoir construction. After a transition period, however, the required reservoir expansion would have to follow that required by the current strategy, but delayed by three years.
6. With the aid of the model the economic feasibility of providing contingency measures to meet critical demand sequences could be studied.
7. The study has clearly demonstrated the value of a simulation model as a tool for the planning and operation of a complex water supply system such as that controlled by the Rand Water Board.

From the results of the study it is evident that no serious fault can be found with the manner in which the Board operates its system and accordingly it was decided that the criteria in current use in respect of the design capacity of pumping plant, pumping mains, distribution pipelines and service reservoirs be maintained.

Co-ordination and co-operation in the provision of service reservoirs
It is clear from the above that both the Board and the consumer, usually the local authority, have to provide storage. In cases where the
Board and the local authority require storage on adjacent sites, which are becoming increasingly difficult to procure, it is obviously in the interests of both parties that one reservoir large enough to meet both their requirements be built. In such cases the Board usually constructs and owns the reservoir with the local authority making a contribution to the cost of the reservoir based pro rata on the storage required.

Strategic storage for Pretoria-Witwatersrand-Vereeniging-Sasolburg (PWVS) region

The PWVS region, although covering only 1,4 per cent of the Republic's area, is an extremely important region from an industrial, commercial and human viewpoint and provision has to be made for storage reservoirs that are strategically placed. In effect the great reservoirs situated at Forest Hill, Meyer's Hill, Merevale and Klipliviersberg with a total capacity of 1 210 M³ equivalent to 16 hours' supply of the entire system, are at such an elevation and so adequately connected to the major supply pipelines of the Board that they can deliver reasonably large quantities of water under gravity to almost any point in the distribution system.

Overall reservoir design

Numerous restrictions may be imposed on the design of a large reservoir by site conditions and service requirements such as capacity, water depth, top water level, etc., all of which must be taken into account in arriving at the optimum solution. The plan shape of the reservoir, i.e. circular, square or rectangular, may be dictated by the size, shape and slope of the site, while the type or types of wall selected will depend on the depth of suitable foundation material and its bearing capacity. In areas of mining or seismic activity reservoirs may be subjected to dynamic loads and should be designed accordingly, as was in fact done in the case of Driefontein reservoir.

Where site conditions or service requirements impose no restriction on plan shape or water depth the optimum economic solution will depend on the relative costs of walls, floor and roof slabs, and of excavation. If excavation necessitates blasting, a shallow basin with high walls and large plan area may be more economical than a deep concrete-lined basin with low walls and smaller plan area. Depending on the location of the site, aesthetics may well influence the final choice of solution.

The Board is conscious of its responsibilities towards the preservation of the environment and particularly in the case of Klipliviersberg reservoir, which is set into a rocky ridge of great scenic beauty, it has done everything possible to restore the construction area to its natural state. Reservoir sites in residential areas are attractively fenced and the grounds laid out and maintained by a qualified horticulturist.

Evolution of the reservoir arrangement

The typical reservoir arrangement has developed over the years from a basic circular tank with flat floor and annular vaulted roof to very large rectangular reservoirs with stepped floors, propped cantilever walls, flat slab roofs and special inlet and outlet structures to ensure good circulation of water, as well as overflow systems to match the ever increasing pipeline capacities. This development can be followed by reference to Table 1.

Development of reservoir components

In the early years sites with suitable founding material and at the required elevation in relation to the area to be served were readily available. Where the reservoir was constructed on the quartzites of the Witwatersrand Geological System good foundation and a sound structure were usually obtained. However, the shales of the System provided less favourable founding conditions and many of the reservoirs constructed on these beds were damaged by settlements arising from the low bearing strength of the soil. Up to about 1958 standard designs for the various components were adopted for each site but during the last 20 years specific wall designs have been prepared to suit low bearing capacity soils.

Floors: In the earlier reservoirs the relatively thin unreinforced concrete floors, usually about 190 mm thick, were cast in a chequer-board pattern but with no provision for expansion or contraction of the concrete. A movement joint was provided at the junction of the wall and floor of the reservoir with a gap filled at first by lime mortar but later by hot-poured bitumen.
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RESERVOIR LINING — FABRIC/BITUMEN
ROAD WIDENING — SEPARATION
With the introduction of roof columns the floor thickness was increased to 230 mm and reinforcement provided in the bottom under the columns for bending and in the top to limit shrinkage cracking. The columns were spayed at the foot to spread the load and were cast on top of the floor to reduce the number of joints to be sealed and to simplify the construction of the floor panels. To facilitate the compaction of the concrete the cone-shaped column bases have now been replaced by cylindrical bases.

In two recent reservoirs founded on rock above the water table, the floor thickness was reduced to 150 mm and the porous concrete drainage layer omitted for the sake of economy. However, problems arose in one of these due to a rise in the water table following two high rainfall seasons while the reservoir was empty for an extended period. The hydrostatic pressure under the floor deflected the edges of the 7.75 m square slabs upwards by an average of 70 mm, with a maximum of 230 mm in one or two cases. As a result of this experience it is now standard practice to provide a minimum floor thickness of 200 mm and a porous concrete drainage layer, whether or not ground water is encountered during construction.

Prior to 1960 floors were level or slightly dished towards a central channel for cleaning and drainage purposes. As the size and depth of reservoirs increased a stepped floor construction was adopted to reduce the height of wall required. Horizontal terrace slabs 300 mm thick were formed to support the roof columns with 230 mm thick sloping slabs between the terraces; the main floor slab at the lowest level was horizontal. Wherever the predetermined floor and top water levels permitted, the reservoir thus approached the economical concept of a concrete-lined basin with a shallow perimeter wall supporting the outer edge of the roof slab.

Walls: Mass concrete walls, 4.4 m deep, reinforced circumferentially with widely spaced old wire rope, were used in the early, small capacity, circular reservoirs but, with the improvement in technology and the need for larger and deeper reservoirs, these were replaced by

Fig 4: Driefontein 124 M^3 reservoir construction showing roof formwork, support trusses and cable-way cranes (1977)

Fig 5: Cross-sections of typical reservoir walls
<table>
<thead>
<tr>
<th>RESERVOIR</th>
<th>CONTRACT NO.</th>
<th>DATE</th>
<th>CAPACITY</th>
<th>DIMENSIONS</th>
<th>WATER DEPTH</th>
<th>CONSTRUCTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Krugersdorp No. 1</td>
<td>100</td>
<td>1910</td>
<td>6.06</td>
<td>45.8 dia.</td>
<td>4.36</td>
<td>Reinforced concrete 600mm thick; wire reinforcement and mild steel reinforcing.</td>
</tr>
<tr>
<td>Signal Hill</td>
<td>168</td>
<td>1918</td>
<td>11.68</td>
<td>61.8 dia.</td>
<td>3.96</td>
<td>Mass concrete tapering from 600 to 1.250 mm thick reinforced circumferentially with steel wire rope.</td>
</tr>
<tr>
<td>Denoi No. 1</td>
<td>169</td>
<td>1920</td>
<td>12.61</td>
<td>61.6 dia.</td>
<td>3.96</td>
<td>Mass concrete tapering from 610 to 1.250 mm thick reinforced circumferentially with steel wire rope.</td>
</tr>
<tr>
<td>Forest Hill No. 1</td>
<td>193</td>
<td>1921</td>
<td>24.15</td>
<td>82.5 dia.</td>
<td>4.57</td>
<td>Mass concrete tapering from 610 to 1.800 mm thick reinforced circumferentially with 25 mm dia steel wire rope.</td>
</tr>
<tr>
<td>Forest Hill No. 2</td>
<td>450</td>
<td>1936</td>
<td>90.21</td>
<td>114.3 dia.</td>
<td>6.84</td>
<td>Mass concrete gravity cast in segments 10 m and 7 m long with 2.2 m and 1.2 m thick, staggered in each lift.</td>
</tr>
<tr>
<td>Krugersdorp No. 2</td>
<td>443</td>
<td>1936</td>
<td>22.71</td>
<td>79.9 dia.</td>
<td>4.6</td>
<td>610 mm to 305 mm thick L shape reinforced concrete.</td>
</tr>
<tr>
<td>Lithozon No. 1</td>
<td>502</td>
<td>1939</td>
<td>11.37</td>
<td>50 x 50</td>
<td>4.6</td>
<td>510 mm to 305 mm thick L shape reinforced concrete.</td>
</tr>
<tr>
<td>Braamfontein</td>
<td>RWD</td>
<td>1937</td>
<td>45.43</td>
<td>61 dia.</td>
<td>8.8</td>
<td>Mass concrete gravity section.</td>
</tr>
<tr>
<td>Roodepoort</td>
<td>RWD</td>
<td>1943</td>
<td>45.43</td>
<td>61 dia.</td>
<td>8.8</td>
<td>Mass concrete gravity section.</td>
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<tr>
<td>Denoi No. 2</td>
<td>RWD</td>
<td>1943</td>
<td>45.43</td>
<td>61 dia.</td>
<td>8.8</td>
<td>Mass concrete gravity section.</td>
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<tr>
<td>Germiston</td>
<td>RWD</td>
<td>1945</td>
<td>90.93</td>
<td>135x67</td>
<td>10</td>
<td>Mass concrete gravity section.</td>
</tr>
<tr>
<td>Forest Hill No. 3</td>
<td>RWD</td>
<td>1946</td>
<td>113.88</td>
<td>120 dia.</td>
<td>10</td>
<td>Mass concrete gravity section.</td>
</tr>
<tr>
<td>Daleside</td>
<td>RWD</td>
<td>1952</td>
<td>46.69</td>
<td>61 dia.</td>
<td>8.8</td>
<td>Mass concrete gravity section.</td>
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<td>Waterval No. 1</td>
<td>806</td>
<td>1955</td>
<td>45.43</td>
<td>61 dia.</td>
<td>8.8</td>
<td>Mass concrete gravity section.</td>
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<td>Seicourt</td>
<td>806</td>
<td>1954</td>
<td>22.71</td>
<td>81 dia.</td>
<td>4.6</td>
<td>- L shape reinforced concrete.</td>
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<td>Modder East</td>
<td>806</td>
<td>1954</td>
<td>11.39</td>
<td>27 dia.</td>
<td>4.6</td>
<td>- L shape reinforced concrete.</td>
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<td>310 mm thick mass concrete slab.</td>
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<td>190 mm thick mass concrete slab.</td>
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<td>230 mm thick mass concrete slab.</td>
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<td>230 mm thick mass concrete slab.</td>
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<td>230 mm thick mass concrete slab.</td>
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<td>230 mm thick mass concrete slab.</td>
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<tr>
<th>RISERVOIR</th>
<th>CONTRACT NO.</th>
<th>DATE</th>
<th>CAPACITY m³</th>
<th>DIMENSIONS m</th>
<th>WATER DEPTH m</th>
<th>WALL</th>
<th>CONSTRUCTION</th>
<th>FLOOR</th>
</tr>
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<tbody>
<tr>
<td>Klipfontein</td>
<td>1040</td>
<td>1940</td>
<td>198,75</td>
<td>148x126</td>
<td>13,4</td>
<td>340 mm thick reinforced concrete sloping with internal buttresses 3m and 2,7m centres; 11,4m and 10,2m long units.</td>
<td>Reinforced concrete slab 230mm thick; 102mm drop panels 3,8m square on 61mm and 64mm dia columns at 7,4m centres. 230mm stone. Stepped - 2x2, 4m terraces 1x2, 3m terrace Horizontal slab 230mm thick Sloping slab 230mm thick 230 mm wide waterstops in 40mm wide expansion joints. Bitumen impregnated filler rubber/bitumen seals. 75mm no fines concrete under floor.</td>
<td></td>
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<td>Langersand</td>
<td>RNB 1958</td>
<td>111,95</td>
<td>111x104</td>
<td>10,7</td>
<td></td>
<td>Tapered mass concrete gravity 11,5 to 6,8m high. plus 380mm thick L-shaped plain and cast concrete cantilever reinforced concrete walls 1,1 to 3,8m high. 10,2 to 15,2m long units.</td>
<td>Reinforced concrete slab 230mm thick; 102mm drop panels 3,8m square on 61mm and 64mm dia columns at 7,4m centres. 230mm stone. Horizontal and 45° sloping slabs 290 mm thick.</td>
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<tr>
<td>Klipviersberg</td>
<td>1164</td>
<td>1974</td>
<td>207,43</td>
<td>232x207</td>
<td>13,4</td>
<td>460 mm thick reinforced concrete cantilever 4,7m high. generally south wall 460 to 220mm thick reinforced concrete cantilever 7,7m high. 15,4m long units.</td>
<td>Reinforced concrete slab 190mm thick 75mm drop panels 3,8m square on 61mm dia columns at 7,6m centres; 150mm stone. Horizontal slab 150mm thick; 45° sloping slab 160mm on rock 290 mm thick on soil. Panels 7,62m square Waterstops: 150mm wide in contraction joints 230mm wide in expansion joints Bresin bonded cork filler in 25mm wide expansion joints, with 35mm polyurethane surface seal (10x10 mm in contraction joints); no-fines concrete layer omitted.</td>
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</tr>
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<td>Sasolburg</td>
<td>RNB 1979</td>
<td>106,50</td>
<td>97 square</td>
<td>10,37</td>
<td></td>
<td>460 to 300mm thick sloping, propped reinforced concrete cantilever.</td>
<td>Reinforced concrete slab 260mm thick 100mm drop panels 3,8m square on 47mm dia columns at 7,75m centres; 150mm stone. Horizontal slab 160mm thick 45° sloping slab 220mm thick Panels 7,75m square Waterstops: 150mm wide in contraction joints 230mm wide in expansion joints Bresin bonded cork filler in 25mm wide expansion joints. 300mm wide butyl rubber surface seals over joints. No fines concrete layer omitted.</td>
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<tr>
<td>Airfield</td>
<td>1266</td>
<td>1975</td>
<td>6x25</td>
<td>67 dia</td>
<td>7,21</td>
<td>500 to 250mm thick Preload post-tensioned reinforced concrete on ring beam on reinforced concrete piles at 5,74m centres (in pairs)</td>
<td>120mm thick reinforced concrete dome. 380mm thick reinforced concrete slab on bored cast in situ 100T 230mm dia reinforced concrete piles at 2,2m centres. 150mm no fines concrete under floor. 230mm wide waterstops in contraction and expansion joints. 230mm thick resin bonded cork filler in 25mm wide expansion joints. Butyl rubber surface seals over joints 240mm wide over contraction joints. 300mm wide over expansion joints.</td>
<td></td>
</tr>
<tr>
<td>Wildebeestjewa No. 2</td>
<td>1266</td>
<td>1978</td>
<td>46</td>
<td>67 dia</td>
<td>13,41</td>
<td>350 to 230mm thick preload post-tensioned reinforced concrete.</td>
<td>350mm thick reinforced concrete dome. 220mm thick reinforced concrete slab 150mm No fines concrete under floor. 250mm wide waterstops in contraction and expansion joints. 230mm thick resin bonded cork filler in 25mm wide expansion joints. Butyl rubber surface seals over joints 240mm wide over contraction joints. 350mm wide over expansion joints.</td>
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</tr>
<tr>
<td>Visksrovein</td>
<td>1207</td>
<td>1978</td>
<td>425</td>
<td>296,75x25</td>
<td>11,19</td>
<td>External walls sloping, propped cantilever. 400mm thick U-shaped division wall incorporating in situ ducts.</td>
<td>Reinforced concrete slab 230mm thick 100m drop panels 3,8m square on 47mm dia columns at 7,75m centres; 150mm stone. Horizontal slab 200mm thick 45° slab 230mm thick Panels 7,75m square 75mm no fines concrete under floor. Outlet ducts provided to improve water circulation.</td>
<td></td>
</tr>
<tr>
<td>Libanon Reservoir No. 2 (extension)</td>
<td>RNB 1955</td>
<td>11,35</td>
<td>50x50</td>
<td>4,6</td>
<td></td>
<td>— shape reinforced concrete.</td>
<td>310 mm thick reinforced concrete flat slab; 90 mm drop panels 2,4m square on 400 mm dia reinforced concrete columns at 6,1m centres; 300 mm soil over 150 mm stone.</td>
<td>230 mm thick mass concrete with bitumen filled joint at wall rebated construction joints.</td>
</tr>
<tr>
<td>Rietvoetsgezicht</td>
<td>855</td>
<td>1957</td>
<td>22,79</td>
<td>57,5 dia</td>
<td>6,8</td>
<td>Mass concrete gravity section.</td>
<td>190 mm thick reinforced concrete flat slab; 90 mm drop panels 3,4m square on 400 mm dia reinforced concrete columns at 6,1m centres; 300mm soil over 150 mm stone.</td>
<td>230 mm thick mass concrete with bitumen filled joint at wall rebated construction joints.</td>
</tr>
</tbody>
</table>
Table 1 — continued

<table>
<thead>
<tr>
<th>RESERVOIR</th>
<th>CONSTRUCT NO.</th>
<th>DATE</th>
<th>CAPACITY M³</th>
<th>DIMENSIONS m</th>
<th>WATER DEPTH m</th>
<th>WALL</th>
<th>ROOF</th>
<th>FLOOR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Waporiende No. 1</td>
<td>RWB</td>
<td>1968</td>
<td>45,43</td>
<td>61 dia</td>
<td>8,8</td>
<td>Mass concrete gravity section.</td>
<td>190 mm thick reinforced concrete flat slab; 50 mm drop panels 2,4 m square on 460 mm dia reinforced concrete columns at 6,2 m centres; 305 mm soil over 150 mm stone.</td>
<td>230 mm thick mass concrete with bitumen filled joint at wall; rebated construction joints.</td>
</tr>
<tr>
<td>Northridge</td>
<td>RWB</td>
<td>1960</td>
<td>90,07</td>
<td>114 dia</td>
<td>8,8</td>
<td>Mass concrete gravity section.</td>
<td>190 mm thick reinforced concrete flat slab; 50 mm drop panels 2,4 m square on 460 mm dia reinforced concrete columns at 6,2 m centres; 305 mm soil over 150 mm stone.</td>
<td>230 mm thick mass concrete with bitumen filled joint at wall; rebated construction joints.</td>
</tr>
<tr>
<td>Waterval No. 2</td>
<td>RWB</td>
<td>1962</td>
<td>61,67</td>
<td>104 x 67</td>
<td>11</td>
<td>340 mm thick reinforced concrete sloping with internal buttresses at 3,1 m centres; 12,5 m long units.</td>
<td>Reinforced concrete flat slab 160 mm thick; 90 mm drop panels 2,4 m square on 460 mm dia columns at 6,2 m centres; 230 mm stone.</td>
<td>Stepped - 3 x 2,1 m terraces 250 mm sloping, 305 mm thick horizontal slabs. Waterstops: 150 mm in construction joints 230 mm in expansion joints 60 mm wide. Bitumen impregnated filler in expansion joints; rubber/bitumen seals. 78 mm no fines concrete under whole floor.</td>
</tr>
<tr>
<td>Wildebeestfontein No. 1</td>
<td>955</td>
<td>1962</td>
<td>22,82</td>
<td>46,6 dia</td>
<td>13,4</td>
<td>320 mm thick Preload post-tensioned reinforced concrete.</td>
<td>100 mm thick reinforced concrete dome.</td>
<td>Reinforced concrete 230 mm thick.</td>
</tr>
<tr>
<td>Mereda</td>
<td>RWB</td>
<td>1964</td>
<td>122,42</td>
<td>111x104</td>
<td>13,4</td>
<td>340 mm thick reinforced concrete sloping with internal buttresses 3m and 2,76m centres; 13,4m and 12,2m long units.</td>
<td>Reinforced concrete slab 230mm thick; 102mm drop panels 3,8m square on 610mm and 665mm dia columns at 7,8m centres. 235mm stone.</td>
<td>Stepped - 3 x 3 m terraces 250 mm sloping, 305 mm thick horizontal slabs. Waterstops: 160mm in construction joints, 230mm in expansion joints 40mm wide. Bitumen impregnated filler in expansion joints rubber/bitumen seals. No fines concrete on soil only.</td>
</tr>
</tbody>
</table>

mass gravity concrete walls, a type used for a large number of reservoirs with wall heights of 8,8 m to 10,1 m during the period 1935 to 1957 (see Fig 5 for cross-sections of walls). In 1938, Chief Engineer Mr M Udwim published a paper [1] which dealt with the design and construction of a 91 M³ concrete reservoir at Forest Hill, with mass gravity concrete walls and a flat slab reinforced concrete roof.

Problems encountered in 1957 with the gravity-type wall due to high heel and toe foundation pressures led to the adoption of a sloping, internal counterfort, reinforced concrete wall to reduce total mass and therefore foundation pressures. Serious failures of two reservoirs, at Blyvooruitzicht and Zwarte Koepjes, and the engineering solutions thereto are set out in paper by Laburn [2].

The reservoir design philosophy developed since then has led to the adoption wherever possible of a concrete-lined basin with sloping sides and horizontal perimeter wall, as shallow as the founding level will permit, supporting the outer edges of the roof slab. Various types of wall have been used, often in the same reservoir, dependent on the allowable foundation bearing pressure and the wall height, ranging from mass gravity and vertical cantilever to sloping, internal counterfort walls. Although the sloping, internal counterfort wall is more efficient than the gravity or vertical cantilever types in reducing toe and heel pressures it proved expensive to construct due to the complicated formwork required and the difficulty of fixing the reinforcement.

This led to the development of an inward sloping, propped cantilever wall of simple cross-section (see Fig 5) which has proved economical in terms of materials, ease of construction and low imposed bearing pressures. The inclined props may be cast in advance of or with the upper part of the wall while the formwork and the reinforcement arrangement are straightforward. The only drawback, compared with a vertical wall, is the longer stripping time imposed by the necessity to support the sloping section until the concrete attains self-supporting strength. This latest type of wall has been used in the Sasolburg 103 M³, Driefontein 124 M³ and Vilkafontein 420 M³ reservoirs recently constructed, the first two departmentally and the last by contract.

An interesting development of the propped cantilever was incorporated in the dividing wall between the two compartments of the 420 M³ Vilkafontein reservoir in Benoni. The vertical wall is propped continuously at mid-height by inclined slabs and the triangular spaces so formed serve as ducts to distribute incoming water over the full width of the reservoir (see Fig 6).

Since 1960 the Board has accepted design and construct tenders for seven circular, small-capacity but deep reservoirs (20 to 46 M³ capacity and up to 14 m wall height) with post-tensioned concrete walls and domed roofs, three of them for pumping surge control purposes. Six of these were designed by Preload Africa (Pty) Ltd using a proprietary system of externally wrapped high tensile steel wire reinforcement protected by pneumatically applied cement mortar, while the seventh incorporated embedded circumferential post tensioning cables.

Fig 6: Cross-section of dividing wall: Vilkafontein 420 M³ reservoir

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Roofs: Roofing was prompted originally by the danger of pollution, when dust and locusts were the main cause of concern. The policy has been a sound one for it excludes the possibility of aerial pollution and makes deliberate interference with the stored water difficult. Moreover, because the reservoirs are insect and vermin proof there is no possibility of mosquitoes and other insects breeding on or in the water.

Roofs, omitted altogether on some earlier reservoirs (but added at a later stage), have evolved from corrugated iron on timber supports (1908) through annular vaulting of reinforced concrete (1910) to reinforced concrete flat slab (1935) and domed (1960) construction. Since 1935 virtually all reservoirs have been covered by a concrete flat slab roof, the exceptions comprising seven smaller capacity circular reservoirs with concrete domed roofs.

Apart from an increase in span from 6.25 m to 7.75 m, a reduction in the thickness of the stone insulation layer from 300 mm to 150 mm and the replacement of plain mild steel by deformed high yield steel reinforcement, the flat slab design has remained virtually unchanged, comparing favourably in both cost and durability with numerous alternatives offered by tenderers from time to time. Concrete domed roofs have proved economic for circular reservoirs within the range of diameter for which formwork is available for hire.

Construction procedures

Walls: The mass gravity concrete walls cast in segments 10.1 m and 7 m long with 2.1 m to 1.2 m long infill sections staggered in each lift to provide a monolithic circular wall without movement joints. Shear keys were provided in each horizontal construction joint and concrete surfaces were chipped before placing the next lift of concrete.

With the transition from circular to rectangular reservoirs and the introduction of the internal counterfort design the wall was built in separate units each equal in length to two floor panels, ie initially 12.5 m and later 15.3 m. A contraction joint with pvc waterbar was provided between wall unit and floor to allow a degree of differential movement between the various elements. The latest propped cantilever and other types of wall are also constructed in units 15.3 m long separated by contraction joints or by expansion joints where necessary in long lengths of wall. Wall units are cast successively rather than alternately to reduce shrinkage movements at the joints.

Floors: In the earlier reservoirs built between 1935 and 1957 floors were cast in panels with rebated construction joints, the only movement joint being that between floor and wall, where a tapered joint was filled with hot-poured bitumen sealed on the surface with cement mortar. A bitumen painted sliding joint was formed between floor slab and top of wall base.

Waterval reservoir, completed in 1962, which features the internal counterfort wall and stepped floor, saw the introduction of contraction joints between floor panels and pvc waterbars in all movement joints. Compressible filler material was used to form the expansion joints and more sophisticated rubberized bitumen was used to seal the movement joints. Porous concrete underfloor and underwall drainagae was also introduced for the first time in this reservoir.

A reasonably smooth floor surface is necessary to facilitate cleaning and power floating has been found to provide an economical and satisfactory finish.

Roofs: Several circular reservoirs built before the First World War were provided with roofs comprising 1.8 m radius semi-circular concrete arches arranged in concentric rings and supported on 685 mm by 610 mm concrete columns at 3 m centres. Timber formwork was used and a very good finish achieved although obviously the cost must have been relatively high. The completed roof was covered with soil to a depth of 300 mm over the crowns of the arches which to this day are clearly defined by the poorer growth of grass on the thinner soil. Krugersdorp No. 1 reservoir was originally roofed with corrugated iron on timber supports, subsequently replaced by annular concrete vaulting as described above.

From 1935 onwards flat slab reinforced concrete roof construction supported on concrete columns with conical heads and bases was adopted as standard. Timber formwork gave way to steel panel forms and gumpole centering to tubular steel scaffolding. A system of tubular steel main and secondary trusses supported from the concrete columns has been in continuous use since 1962 on reservoirs constructed departmentally, by the Board, proving most economical for the common roof heights of 14 m and more.

In order to minimize the effects of shrinkage, flat slab roof construction is commenced at one corner of rectangular reservoirs and is advanced along a diagonal front to the opposite corner. The roof slab is cast in panels monolithically between widely spaced expansion joints, with reinforcement continuous through rebated construction joints, the conical column heads being poured with the drop panels. A fall towards the edges of the roof is provided for

Fig 7: Aerial view of Meyer's Hill 273 M³ reservoir construction (1955)
drainage but still allows curing by ponding. Deformed, high-yield steel reinforcement is used throughout to control thermal and shrinkage cracking. The outer edges of the roof slab are supported on sliding bearings on top of the wall to prevent transfer of indeterminate horizontal loads into the relatively flexible wall of the latest design.

**Materials**

**Concrete:** Coarse and fine aggregates are specified in accordance with SABS 718; cement may be ordinary Portland, Portland blast furnace or a 50/50 mixture of ordinary Portland and milled blast furnace slag, the last being most commonly used. However, in the roof, where early strength is required to release formwork, ordinary Portland cement only is used.

Structural concrete is specified by 28-day compressive strength with a minimum cement content to ensure impermeability and durability. This latter requirement may lead to excessive early strengths and high shrinkage rates where the sand used has a low water demand. The concrete mix is usually designed by a specialist organization using the materials selected for the project but is often modified in the light of site experience. Pozzolanic type additives have been used to improve workability or to increase strength.

**Reinforcement:** In recent years plain round mild steel reinforcement has been replaced by deformed high yield steel with considerable saving in cost, due partly to its higher tensile strength but mainly to its superior bond characteristics, qualities which can be used to great advantage when design is carried out in accordance with the current code of practice for water retaining structures, BS 5337. In general, small diameter bars are used, as recommended in the code, for optimum crack resistance.

**Waterbars:** Pvc waterbars were provided in joints for the first time in 1960, in Waterval reservoir. 150 mm wide in contraction joints and 230 mm wide, with a centre bulb, in expansion joints. This practice still applies to all movement joints. Fig 9 shows typical details of movement joints in walls and floors.

**Joint fillers:** When waterbars were introduced in movement joints the gap required was formed by strips of bitumen-impregnated fibrous board that were left in place and tended to deteriorate with time in the presence of moisture. This problem has been overcome by the substitution of a resin-impregnated granular cork material that is rot resistant and has better recovery characteristics after deformation.

**Sealants:** Joint sealants fulfill two primary functions, firstly in preventing the ingress of dirt which might support bacterial growth and secondly in forming a first line of defence against leakage through the joint, backed up by the waterbar itself. The sealing material in the first movement joints, those between floors and walls, was a hot-poured bitumen that was used without a primer and could be easily run into horizontal joints. After cooling the bitumen filling was usually protected by filling the top 25 mm of the joint with cement mortar. Here the bitumen sealant was the only barrier to leakage through the joint.

In 1960, in addition to the expansion joint between floor and wall, contraction joints were introduced between floor panels and between wall units, and waterbars were provided in all joints. Surface primers and rubberized bitumen sealants, hot-poured for horizontal joints and cold-applied in putty form for vertical joints, were introduced at the same time. Good adhesion to the concrete was difficult to achieve and accidental overheating often adversely affected the limited plasticity of the hot-poured material. A further disadvantage proved to be the extrusion of the material by hydrostatic pressure through holes in damaged waterbars.

By 1970 elastomeric compounds had become available, offering improved adhesion to concrete, tensile strength and elasticity as well as long life. One of these, a two-part polyurethane, was used with success in Klipriviersberg reservoir; the preparation and priming of concrete contact surfaces was of critical importance and despite the closest supervision a significant percentage of joints required rescaling before being accepted. Moreover the concrete had to be completely dry at the time of application to ensure satisfactory adhesion. Correct proportioning and thorough mixing of the two constituent parts without air entrainment were equally vital to the achievement of good results.

More recently, after extensive testing and subsequent use in the repair of joints in a large reservoir, a surface applied bandage-type seal of laminated butyl rubber strip (see Fig 9) has been used with good results. Advantages include a longer water percolation path around the seal (usually 230 mm in width and placed centrally over the joint), adhesion to a large area of sound concrete clear of the often weaker concrete at the joint, good mechanical strength, flexibility and ease of inspection and repair. The formwork to edges of contraction joints is also simplified by the elimination of the recesses required for other types of sealant.

The contact surfaces of the concrete are mechanically abraded to remove laitance and ensure good adhesion to sound concrete. Adhesion to a correctly prepared concrete surface is excellent — in severe pull-off tests the rubber seal will delaminate before it fails in adhesion. In expansion joints a metal backing strip prevents deflection of the seal under hydrostatic pressure. This type of joint seal has been used with success in three of the most recently constructed service reservoirs.

**Sub-floor drainage**

Sub-floor drainage has two importance functions, viz to prevent a build-up of hydrostatic pressure under the floor (whether due to ground water or to leakage water) when the reservoir is empty and to provide an indication of any leakage. The typical sub-floor drainage system comprises a 75 mm thick layer of porous (no-fines) concrete under all structural concrete, draining into a network of perforated collecting pipes that discharge into external manholes or, as in the case of Viaklofontein, into an access and inspection tunnel running the full length of the reservoir. The latter arrangement has the advantage that leakage points can be readily located for remedial treatment. For the same reason the subfloor drainage system of circular reservoirs is divided into a number of sectors, each discharging into a separate external manhole.

**Watertightness**

The Board’s specification requires that on completion a reservoir be filled slowly, allowed to stand for 10 days to allow for absorption of water by the concrete and then tested for watertightness by measuring the drop in level over a period of 72 hours. The permissible drop in level is 3 mm per 24 hours (21 mm in seven days); if this is exceeded the reservoir is emptied and the contractor required to carry out the necessary remedial work. The BS 5337 recommendation is, however, more severe, requiring a maximum drop in level of only 10 mm in seven days.

Almost invariably any leakage that may occur has been traced to joints of one kind or another, involving displaced or perforated waterbars, poor adhesion of sealing materials or, less frequently, in the case of construction joints, careless preparation or cleaning of the concrete joint surfaces. Continuous, strict supervision is essential over long periods to ensure uniformly good results, especially where, in the larger reservoir, the total length of movement joints alone may exceed 5 km.
Pipework arrangements
Apart from the basic requirements of structural stability and watertightness possibly the most important factor in the satisfactory operation of a service reservoir is the achievement of good water circulation that will prevent the formation of dead or stagnant areas of poor quality water. Many of the earlier and smaller reservoirs have a single inlet/outlet pipe that restricts circulation within the reservoir but this is compensated for by the large daily drawdown during peak demand periods when the reservoir is often virtually emptied.

This does not necessarily apply to some of the larger reservoirs where under normal operating conditions the daily fluctuation in level may be relatively small over prolonged periods. In such cases internal circulation becomes important and has been improved usually by locating the inlet and outlet sumps as far apart as possible, preferably on opposite sides of the reservoir, although cost too has been a deciding factor in certain cases. Where problems of water quality have been encountered in reservoirs with a single inlet/outlet pipe an additional inlet or outlet has been provided to improve circulation.

Vlakfontein is the first reservoir to incorporate special features, other than widely separated inlet and outlet sumps, to ensure good circulation of the water. Inlet distribution ducts form part of the division wall (see Fig 6) between the two compartments of the reservoir with circular ports over their full length that ensure movement of the whole body of water towards two outlets at the opposite end of each compartment. A transverse, above floor, open ended collecting duct discharges into a central underfloor outlet tunnel leading to the outlet control valve chamber to the south of the reservoir. The arrangement in the south compartment is similar, with the exception that a short length of steel pipe replaces the concrete tunnel. A concrete outlet tunnel proved to be more economical than a concrete encased steel pipe of equivalent capacity and, moreover, allowed the addition at minimal cost of an inspection tunnel in which the discharge from the various sections of the underfloor drainage system can be monitored.

Overflow pipes, an essential feature of all reservoirs, were until recently generally of small diameter and discharged a short distance away from the reservoir site, either onto unoccupied land or, in built-up areas, into the local stormwater drainage system. However, with the major increase in pumping capacity and in the diameter of pumping mains in recent years the maximum rate of overflow has correspondingly increased to the point where the overflow water can safely be discharged only into the nearest well-defined watercourse, by means of a pipeline that may be several kilometres in length and costs R250 000, as was the case at Vlakfontein.

In most reservoirs the overflow comprises a vertical steel pipe of up to 1 500 mm diameter, encased in concrete for rigidity, with a bellmouth inlet. To avoid the complication of passing through the wall, the overflow pipe usually passes under the wall footing in a concrete encasement. At Vlakfontein it was necessary to incorporate rectangular overflow chambers in the dividing wall with weirs long enough to accommodate the maximum overflow rate, without overtopping the 500 m freeboard of the walls. It is now the practice to negotiate a servitude for the overflow pipe route at the time of purchasing the reservoir site.

The size (often exceeding 2 m diameter) and depth of inlet and outlet piping is often such that major earthworks are necessary for their installation, requiring careful integration into the overall construction programme, particularly on a restricted site or where the use of explosives is required for excavation, while the valve chambers associated with the pipework are themselves impressive in both size and cost.

Cost of reservoir construction
The costs given in Table 2 include the cost of pipework, valve chambers, etc but exclude the cost of land. As would be expected the unit cost of storage decreases as the capacity of the reservoir increases.

Future trends
As industrial and residential development advances in all directions the choice of suitable reservoir sites becomes ever more limited, with implication that sites may have to be accepted in future that are less than ideal in many respects such as elevation, location and suitability of founding material. Poor founding material in particular will seriously affect the cost of construction, especially if piled foundations are necessary.

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**Movement joints in reservoir floor slab**

1. **ONE COAT BONDBREAKER APPLIED TO SURFACE**
2. **25mm THICK RESIN IMPREGNATED CORK FILLER**
3. **250mm PVC WATERBAR WITH CENTRE BULB**
4. **80mm WIDE X 16g ALUMINIUM BACKING STRIP WITH 110mm WIDE X 375 MICRON PVC SHEET BETWEEN CONCRETE & STRIP**

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**Movement joints in reservoir walls**

5. **350mm WIDE BUTYL RUBBER SEALING STRIP**
6. **75mm THICK POROUS CONCRETE**
7. **50mm THICK BLINDING CONCRETE**
8. **240mm WIDE BUTYL RUBBER SEALING STRIP**
9. **250mm PVC WATERBAR (PLAIN)**

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**Fig 9: Typical details of movement joints in floors and walls**
Table 2: Cost of service reservoirs

<table>
<thead>
<tr>
<th>Reservoir</th>
<th>Put into service</th>
<th>Capacity M1</th>
<th>Cost R</th>
<th>Cost R/kl</th>
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<td>Brakpan</td>
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<td>45</td>
<td>80 000</td>
<td>1,76</td>
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<td>Daleside</td>
<td>1952</td>
<td>45</td>
<td>213 000</td>
<td>4,69</td>
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<td>1955</td>
<td>45</td>
<td>231 000</td>
<td>5,13</td>
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<td>Meyer's Hill</td>
<td>1956</td>
<td>273</td>
<td>825 000</td>
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<td>45</td>
<td>252 000</td>
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<td>46</td>
<td>242 000</td>
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<tr>
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<td>23</td>
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<tr>
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<td>62</td>
<td>411 000</td>
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<td>Meredale</td>
<td>1964</td>
<td>122</td>
<td>712 000</td>
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<td>1968</td>
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<td>50</td>
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<td>1 730 000</td>
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<td>4 600 000</td>
<td>10,92</td>
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<td>46</td>
<td>980 000</td>
<td>21,33</td>
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<td>Sesolburg</td>
<td>1979</td>
<td>103</td>
<td>1 100 000</td>
<td>10,64</td>
</tr>
</tbody>
</table>

* Circular with domed roof and post-stressed wall.  
+ Wall and floor supported on bored, cast in situ concrete piles.

The size of reservoirs has shown a steady and occasionally a dramatic tendency to increase in step with the rising demand for water; fortunately the unit cost of storage capacity varies inversely with the size of the reservoir and spiralling costs can to some extent be offset by the provision of large reservoirs with standardized wall, floor and roof construction involving maximum utilization of formwork and repetition of operations.

If future projections are correct and the demand for water within the Board’s area of supply is in fact quadrupled by the end of the century, it is certain that great challenges lie ahead for the Board’s civil engineers in the design and construction of service reservoirs to keep pace with this rapid expansion.

Acknowledgements
Permission from the Chairman of the Rand Water Board to publish this paper and the assistance of Mr A D Hardwick are gratefully acknowledged.

References
2. Laburn, P.J. Difficulties encountered in the design and construction of service reservoirs of the Rand Water Board. 4th Quin Conv, SA Instn Civ Engrs, 1963.

Call for nominations and applications for SAICE awards for meritorious research

In terms of a Council Resolution, dated 28 May 1968, the Research and Development Committee of the SAICE hereby invites nominations or applications for consideration for awards for meritorious research in the field of civil engineering.

Nominations and applications should reach the Secretaries of the SAICE, PO Box 61019, Marshalltown, 2107, not later than 31 August 1981.

These awards are regarded as very special awards and will only be awarded for research of outstanding quality.

Conditions of award

During 1968 (Council resolution 28 May 1968 and EXCO meeting 14 September 1973) Council introduced an Award for Meritorious Research in order to acknowledge the role of research in the field of civil engineering and to encourage outstanding research work.

The Award for Meritorious Research is to be made in accordance with the following rules and procedures:

1. This Award is for meritorious civil engineering research and is open to all persons in all areas in South Africa but need not necessarily be made to a member of the SAICE or a civil engineer. The Award can be made to an individual or a team.

2. Nominations together with motivations are invited annually from individual members, Branches and Divisions and are to be submitted through the Secretaries of the SAICE to the SAICE’s Research and Development Committee which will make its recommendation to Council. A particular research work could be brought to the notice of the Research and Development Committee via either of the two channels, viz:  

2.1 An application for consideration may be sent directly by the person or team who has done the research. The application must be accompanied by a copy or an abstract of the work and, if possible, the names of possible referees. Persons who assisted must be mentioned and the degree of assistance must be stated.

2.2 The nomination of a research worker and his or her work by another corporate member of the SAICE who must furnish a brief motivation for the recommendation and the present address of the recommended person. The recommended person must agree to be considered and will be asked by the R & D Committee to submit details as in 2.1 above.

3. All nominations and applications for consideration must be received by the Secretaries of the Institution not later than 31 August of a particular year, to be considered for Award to be made at the Annual General Meeting in March of the following year.

4. The list of nominees and details of their work will be circulated to all members of the Research and Development Committee and, if considered necessary, to other referees also. Committee members and/or other referees will be asked to consider whether they are satisfied that the research projects entered are sufficiently outstanding to warrant further consideration, especially with regard to each of the following criteria:

4.1 Relevance to the profession  
4.2 Practical benefit  
4.3 Ingenuity  
4.4 High academic merit

The length of the work will not be major consideration. Preference will be given to published work which has been open to criticism, eg in the technical press, a recognized journal or a conference. In this context, a thesis will also be regarded as published work. Research work which is confidential and therefore not available for further publication, will not be considered for an award.

The candidates will not be considered as being in competition with each other and each research project will be considered on its own merits. Consideration of research projects will not be restricted to current research and awards may be made retrospectively.

5. After due consideration, the Research and Development Committee will draw up their list of Nominees for the Award for Meritorious Research and submit this confidential list, together with a short motivation for each case, to Council.

6. The presentation of the award shall be made by the President or his representative at the Annual General Meeting or on another suitable occasion. On each occasion the number of awards will not be limited but the right to make no award will also exist.

7. The Secretary shall cause to be maintained at the Institution’s Head Office a list of recipients of the award giving the date of the award and a short citation detailing the reasons for the award.