Saving a doomed reservoir

Erhard Kruger*(AI) and Marelize Mostert** describe the rehabilitation of Waterval Reservoir, Roodepoort, Johannesburg in South Africa

The Waterval reservoir, with a capacity of 45.5M litres is situated in Florida, Roodepoort, a suburb of Johannesburg, South Africa. Structural drawings are dated 9 February 1953, suggesting that the reservoir was constructed during 1953 and 1954. The reservoir is owned and operated by Rand Water, which supplies potable water to Pretoria, Johannesburg and environs.

During the 1990s large cracks developed in the reservoir wall. With time, the width of these cracks increased, resulting in some of the cracks being up to 20mm wide (Fig 1). Various unsuccessful attempts were made to seal these cracks with sealants and bandages. Eventually the reservoir had to be decommissioned in 2001 due to excessive leakage and concerns regarding its structural integrity, situated adjacent to a residential area.

In May 2006 Rand Water called for tenders and proposals to rehabilitate the reservoir, and Nyeleti Consulting, with HGK Consulting CC as specialist sub-consultant, was awarded the tender.

The reservoir is circular in shape. It has a gravity type mass concrete wall and reinforced concrete floor, roof and columns.

The wall has a fairly complicated section (see Fig 2). Over the top 1.1m, the wall thickness is 609mm. Over the next 5m, the wall thickens to 3m. Over the next 1.2m it thickens to 3.8m, and over the next, 1.2m to 5.5m. Over the bottom 1.2m the thickness remains constant. The total internal wall height is 9.45m and the internal diameter of the wall is 81.075m. On the inside of and at the bottom of the wall there is a toe of 610mm x 610mm supporting the floor slab. Only the top 1.1m portion of the wall is reinforced.

The 190mm thick reinforced concrete roof slab is supported by 120 columns on a grid of 6.25m x 6.25m. Drop panels, 2.44m x 2.44m x 90mm thick are provided over the columns. The 460mm diameter columns have conical column heads and footings.

The floor slab is 230mm thick and is level. Joints are supplied at 6.25m intervals halfway between column centre lines. Along the centre line of the reservoir a 1200mm wide scour channel, sloping towards the scour valve and overflow pipe, was provided.

The reservoir is surrounded by an earth embankment, the level of which is approximately 1m below the top of the reservoir wall. Due to the slope of the terrain, the height of the fill varies along the perimeter, with the embankment being at its highest on the southern side of the terrain, where the reservoir is at its closest to the residential area and where the largest cracks in the wall also occurred.

Existing pipework

Water is supplied to the structure through a 740mm diameter steel gravity pipeline. The pipe splits into 450mm diameter duty and 635mm diameter emergency inlets, each with a control valve situated in an inlet chamber before entering through the mass concrete wall at a level 1700mm above the reservoir floor.

Two 800mm diameter steel bottom outlet pipes are located next to each other slightly recessed into the floor and encased in mass concrete. Only one of the outlets was operational.

A vertical 400mm diameter bell-mouthed steel pipe allows for overflow water to be drained from the structure. At the bottom it is connected to a scour valve. The scour/overflow pipe extends below the reservoir floor to a point beyond the embankment.

Rehabilitation concept

Both the strength and serviceability of the reservoir had to be addressed. It was apparent that the reservoir wall had insufficient

1 Cracks in the reservoir
2 Complex cross section of the wall

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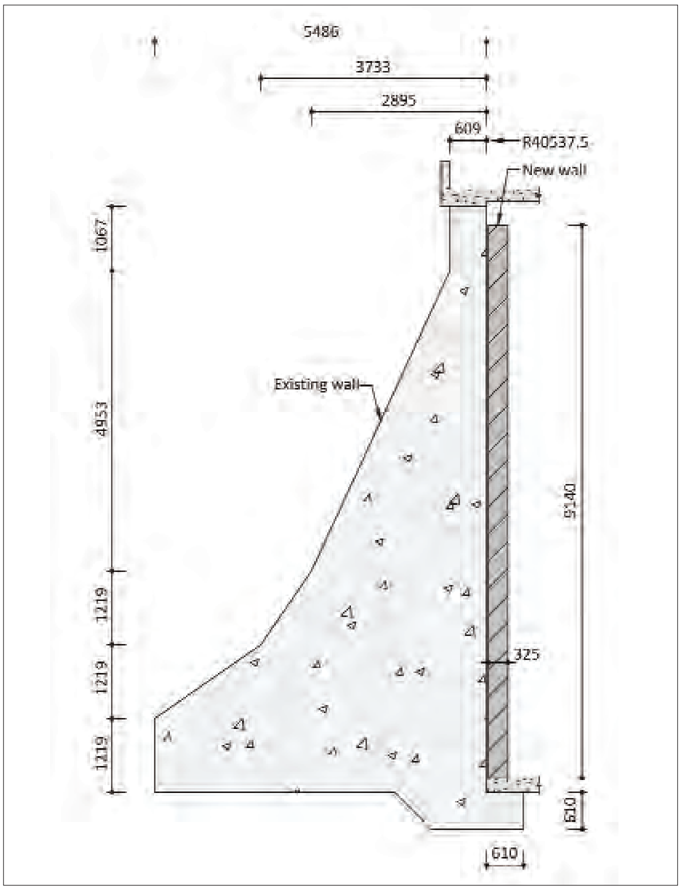

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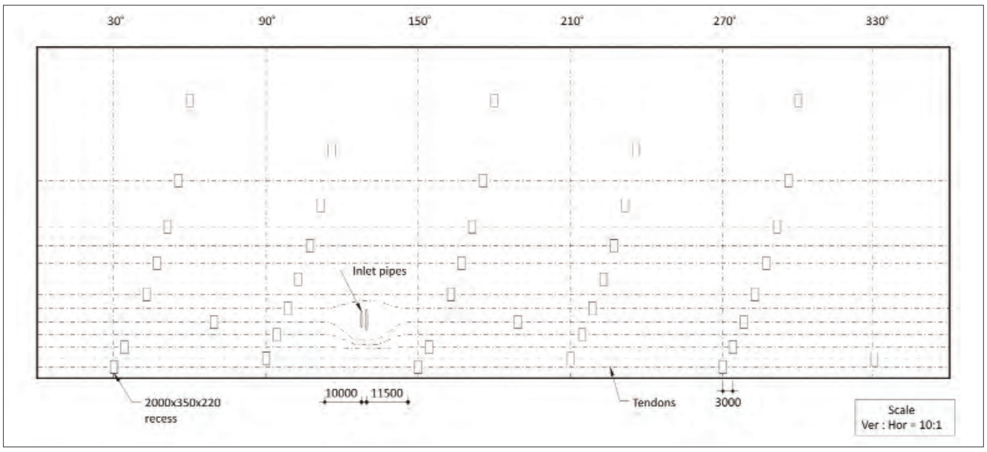

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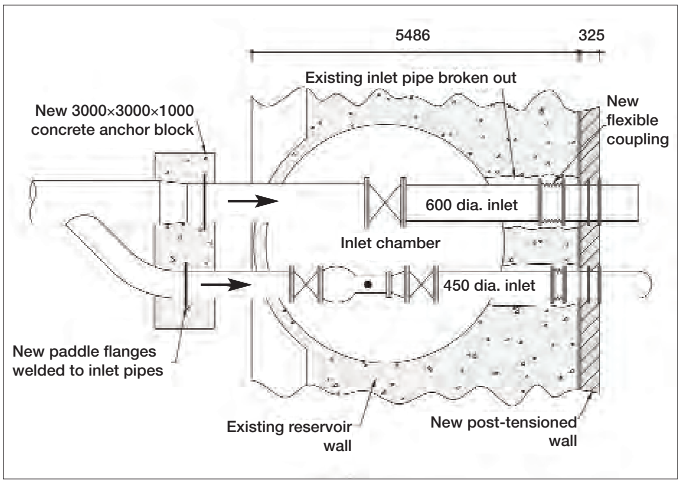
3 Temporary recess for the wall jacks

4 New inlet pipes and flexible couplings



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strength, which resulted in the wall cracking.

The possibility of post-tensioning the reservoir wall by removing the embankment and installing tendons on the outer face of the wall, was investigated. This possibility was ruled out quickly due to the following:

– the post-tensioning force required to overcome friction between the wall and the ground was excessive;

– due to the inclined outer surface of the wall, it would have been difficult keeping tendons in position vertically, due to the component of the pre-stressing force parallel to the sloped surface which would have caused the tendons to slip upwards;

– removal and re-instatement of the embankment would have been costly.

The inadequate strength of the wall was addressed by constructing a new post-tensioned concrete wall on the inside of the existing reservoir wall. Since the water-tightness of the reservoir floor was also suspect, it was decided to line the whole reservoir, including the new wall, to achieve water-tightness.

Wall design

The following had to be taken into account in the wall design:

– avoiding buttresses, which are normally provided for post-tensioning, to simplify installation of the liner;

– the presence of two inlet pipes located in the wall approximately 1.7m above floor level which had to pass through the new wall;

– simplifying construction due to working in a confined space and with the existing roof intact.

The new wall, which is separated from the existing wall with 25mm thick closed cell expanded polyethylene joint former, is 325mm thick and 9.14m high. The full water depth is 8.45m above floor level. The top of the new wall is therefore 400mm below the soffit of the roof slab. The wall is supported on a 9t/m Teflon sliding bearing. The concrete strength is 45MPa/19mm. To avoid buttresses on the inner wall surface, the VSL type Z intermediate type post-tensioning system was used. Each tendon consists of 12, 12.9mm diameter low relaxation strands. Temporary recesses, 2000mm long x 350mm high x 220mm deep, were provided in the wall for the jacks (see Fig 3). Three stressing points, spaced at 120° on the circumference of the wall, were provided on each tendon level. The stressing points on every alternate tendon were staggered. Tendons clashing with the two inlet pipes were displaced vertically locally at the pipe positions. Recesses at each stressing point were staggered by 3m (measured centre to centre) on the circumference. After post-tensioning of tendons the recesses were filled with a high strength non-shrink grout.

Since the reservoir wall is free sliding and therefore moves inwards during post-tensioning, and outwards under hydrostatic load, inlet pipes protruding through the wall had to be modified. The existing 635mm and 460mm diameter pipes were broken out from the face of the existing wall to the inlet chamber located within the wall. Concrete was demolished to allow a 50mm clearance around pipe flanges. New inlet pipes, with puddle flanges being cast into the new wall, and flexible couplings located within the existing wall, were installed (see Fig 4). On the outside of the existing wall, puddle flanges were welded to the inlet pipes, and the pipes anchored in a new 3m x 1m x 3m concrete anchor block.

Liner

A 2mm thick ethylene vinyl acetate (EVA) co-polymer was chosen as liner, since it is elastic, flexible and tough, with excellent puncture and tear resistance, and tested and approved for use in potable water structures.

The floor slab, wall and columns (including column footings and column heads) were lined. On both the floor and wall the EVA liner was underlain by a perforated HDPE heat formed multiple cuspated drainage sheet for drainage and leakage detection purposes. The surface areas of liner on the relevant structural elements were as follows:

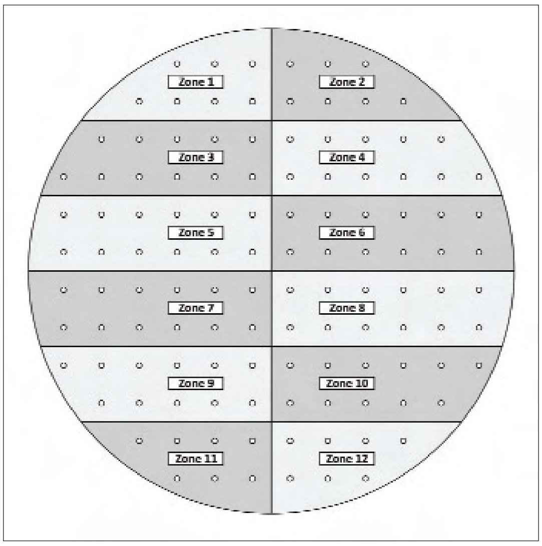
– floor slab: 4860m^2

– wall: 2300m^2

– columns, column footings and column heads: 2100m^2.

Due to the large area being lined, the reservoir was divided into 12 zones to facilitate easier detection of leaks (see Fig 5), each zone containing a segment of the floor and wall, as well as a number of columns.

Zones were separated by fixing the liner to the floor slab and wall on the borders of the zone with double-sided adhesive tape. The existing scour channel on the centre line of the reservoir was changed to a drainage channel. Mass concrete separation strips coinciding with the borders of drainage zones, were cast across the width of the drainage channel, as well as along the centre of the channel, thus dividing the channel into 12 sections. At the lower end of each section of the channel, a perforated 32mm diameter HDPE pipe was installed upslope from and against the separation strip running across the channel, to collect any leakage



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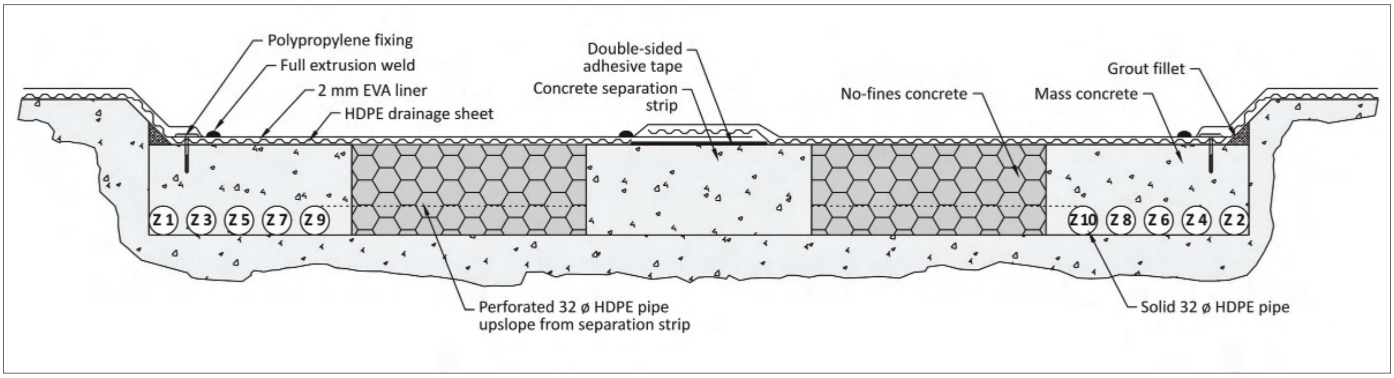
water from the particular zone. The perforated pipe was connected to a solid 32mm diameter HDPE pipe with a 90º bend to drain leakage water (see Fig 6). The solid pipes were encased in mass concrete. No-fines concrete was cast between separation strips and concrete encasement in the channel. The leakage detection pipes were routed through the redundant (additional) outlet pipe into the outlet chamber, where they were clearly marked to indicate the zone being served by each pipe.

In collaboration with the lining sub-contractor, Aquatan Lining Systems, details for the fixing of the liner were developed. It was important that the liner at the connection of the wall to floor slab would be able to accommodate the expected 15mm outward movement of the wall under hydrostatic load. It was also important to ensure continuity of the HDPE drainage layer on the wall surface to the floor surface. The detail as shown (see Fig 7) was developed to allow for the above-mentioned.

The ladder, which was previously fixed to the reservoir wall, was replaced with a square galvanized steel lattice structure. A reinforced concrete slab of 1600mm x 1200mm x 300mm thick, to which the bottom of the ladder was fixed, was cast on a sacrificial layer of 2mm EVA on the installed floor liner. The only other fixing of the ladder was to the roof slab at the top. This prevented any protrusions through the liner on the wall, which are notoriously difficult to make watertight.

A reinforced concrete stilling basin, also being separated from the installed floor liner by a sacrificial layer of 2mm EVA, was cast

5 Leak detection zones
6 Pipe connection to drain leakage water
7 Detail of liner connection of wall to floor slab



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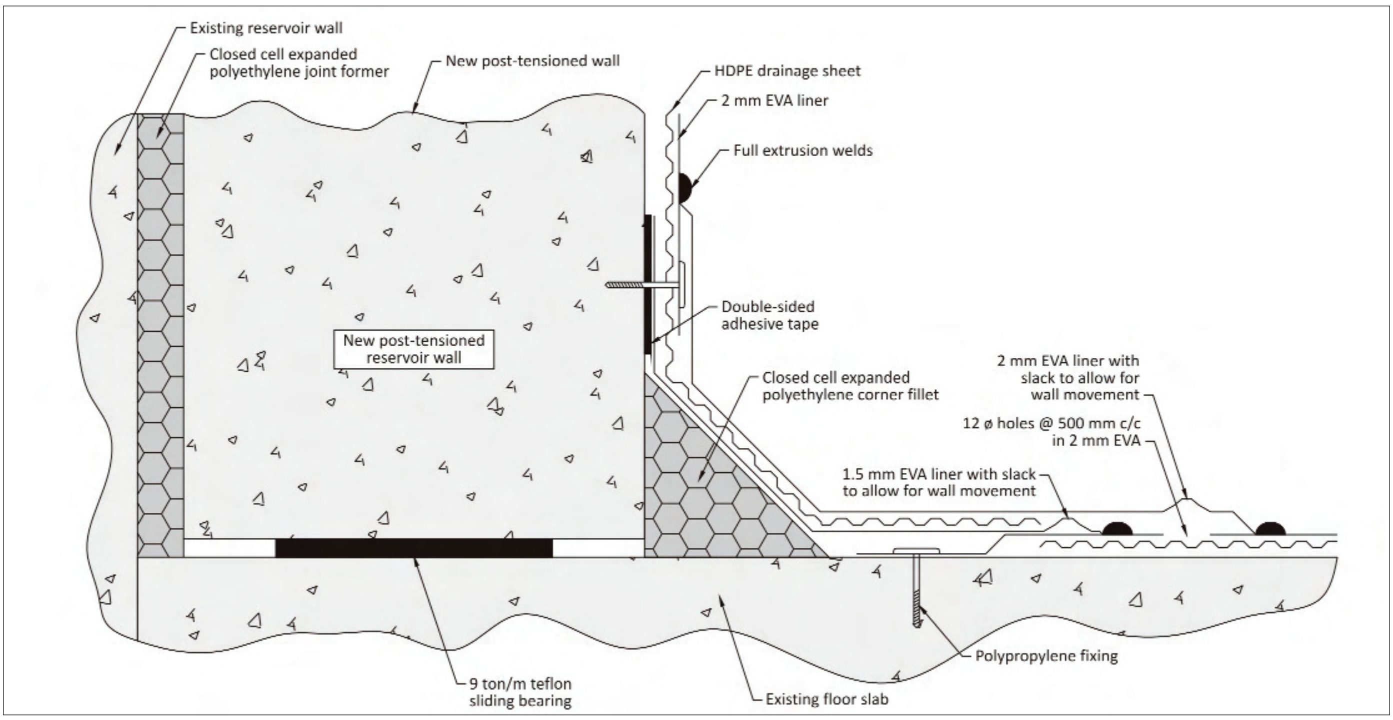

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8 RC concrete distilling basin
9 Wall formwork fixed to the tie rods
10, 11 Post tensioning of the wall
12 View of the lined reservoir

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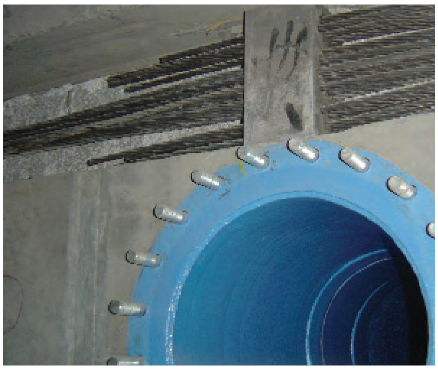

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at the inlet pipes to prevent horizontal displacement of the liner during filling of the reservoir, which would exert undesirable forces on the wall to floor liner connection (see Fig 8). In addition, baffle plates were added to the inlet pipes to dissipate energy during filling, thus protecting the liner.

A template of the developed surface of the conical column heads and footings were used to cut the liner for these elements. The liner was then folded around these elements and welded using special EVA extrusion welding.

The inlet, outlet, overflow and scour pipes had to protrude through the liner. This was facilitated by clamping the liner between two flanges and using neoprene gaskets to ensure water-tightness.

Construction

Construction provided several challenges to the contractor, Stefanutti & Bressan Civils (Pty) Ltd (currently named Stefanutti Stocks (Pty) Ltd). Keeping formwork for the wall in place and casting of the wall necessitated special measures. Nuts for tie rods of the wall formwork were attached using epoxy resin (glue) into the existing reservoir wall. Tie rods were screwed in, ferrule pipes of the required length installed over the tie rods, and the wall formwork fixed to the tie rods (see Fig 9). After the wall had been cast, the tie rods were removed. To simplify casting of the wall, temporary holes were made at regular intervals along the perimeter of the roof slab directly above the new wall (Fig 9), and concrete for the wall was pumped from outside the structure.

Post-tensioning of the wall went smoothly. Since post-tensioning on each level comprised three tendons, three stressing jacks were employed at each level simultaneously, as each individual tendon is stressed against the neighbouring tendon (see Figs 10,11).

End result

The allowable leakage rate was specified as 10 litres/min at the full water depth of 8.45m. As is normally the case with lined reservoirs, the leakage rate at first testing exceeded the allowable. With a water head of 2m, the rate was 3.3 litres/min, and with a 4m head, 5.1 litres/min. The reservoir was then emptied, the liner thoroughly inspected especially in the zones where leakages were recorded, and spark testing done on the liner. Some mechanical damage and pinholes were found, and anything suspicious was repaired. A leaking valve was also found and repaired. Eventually, the leakage rate at a water head of 8m was 8 litres/min. Considering the total area of liner installed was 9260m^2, the result, which is about three times the allowable leakage rate of a newly constructed concrete reservoir, was satisfactory. A view of the lined reservoir is shown in Fig 12.

Conclusion

The cost of the rehabilitation of the reservoir was approximately 30% of the cost of a new 45.5M litre reservoir. In the process of rehabilitation about 2.5M litres (or about 6%) of the original capacity was sacrificed, due to the lowering of the full water level and encroachment of the new wall into the storage space. The guaranteed service life of the rehabilitated reservoir is 20 years, which is the guarantee period on the liner. Previous experience of the performance of EVA liners, however, indicates that the service life would probably be in the order of 30 years. This reservoir proves that rehabilitation should be considered as an option with problem reservoirs.

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